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NATIONAL DAM SAFETY PROGRAM. PINFS LAKE DAM (NJ00250), PASSAIC --ETC(U)
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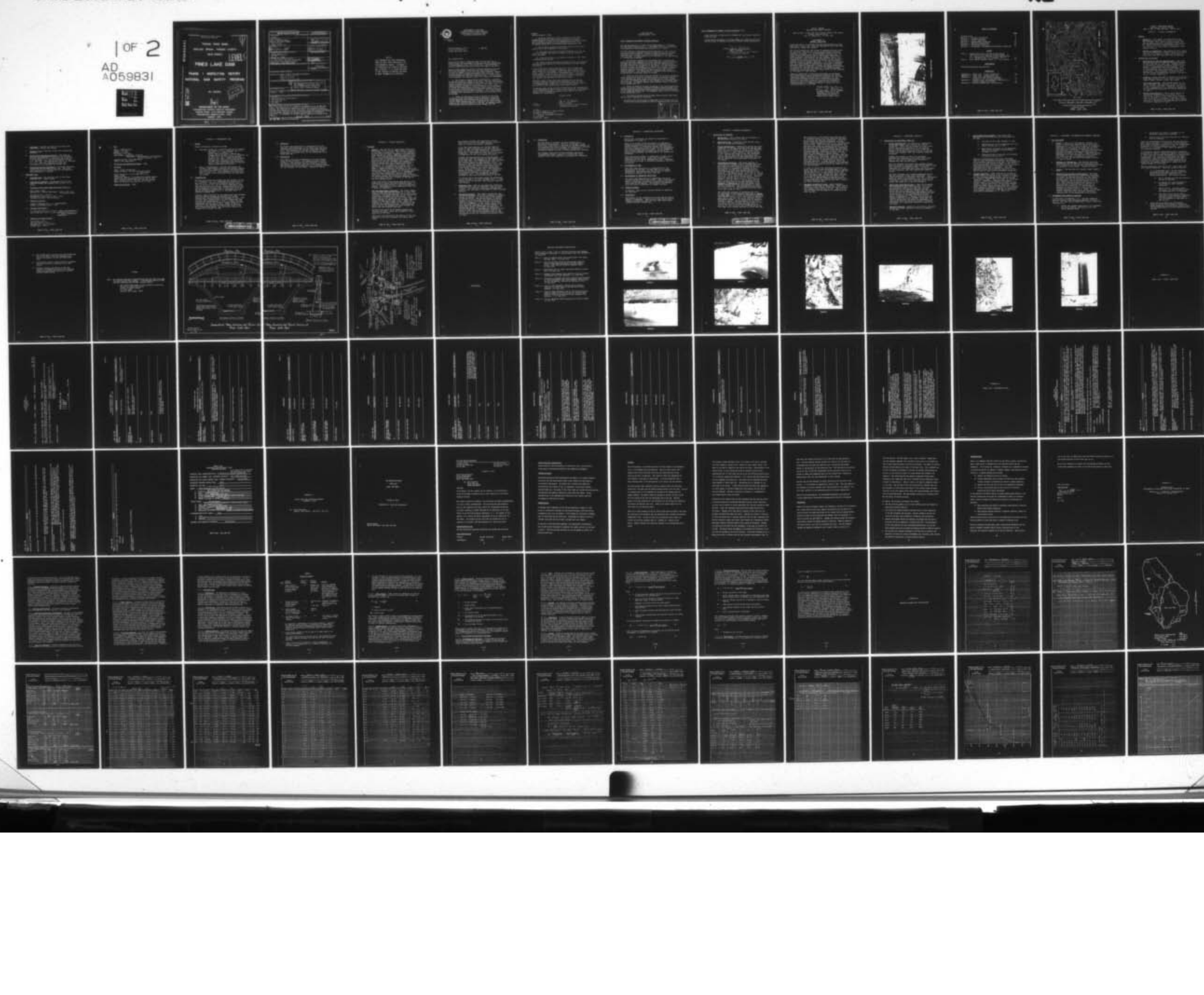

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PASSAIC RIVER BASIN

HAYCOCK BROOK, PASSAIC COUNTY

NEW JERSEY

LEVEL II

PINES LAKE DAM

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

DDC FILE COPY

NJ 00250

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OCT 10 1978
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DEPARTMENT OF THE ARMY
PHILADELPHIA DISTRICT, CORPS OF ENGINEERS
CUSTOM HOUSE - 2D & CHESTNUT STREETS
PHILADELPHIA, PENNSYLVANIA 19106
AUGUST 1978

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SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

REPORT DOCUMENTATION PAGE

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18. SUPPLEMENTARY NOTES

Copies are obtainable from National Technical Information Service, Springfield, Virginia, 22151.

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

Dams--N.J.
National Dam Safety Program Phase I
Pines Lake Dam, N.J.
Dam Inspection
Dam Safety

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

This report cites results of a technical investigation as to the dam's adequacy. The inspection and evaluation of the dam is as prescribed by the National Dam Inspection Act, Public Law 92-367. The technical investigation includes visual inspection, review of available design and construction records, and preliminary structural and hydraulic and hydrologic calculations, as applicable. An assessment of the dam's general condition is included in the report.

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DEPARTMENT OF THE ARMY
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PHILADELPHIA, PENNSYLVANIA 19106

IN REPLY REFER TO
NAPEN-D

5

Honorable Brendan T. Byrne
Governor of New Jersey
Trenton, New Jersey 08621

19 SEP 1978

410 795

Dear Governor Byrne:

Inclosed is the Phase I Inspection Report for Pines Lake Dam in Passaic County, New Jersey which has been prepared under authorization of the Dam Inspection Act, Public Law 92-367. A brief assessment of the dam's condition is given on the first three pages of the report.

Based on visual inspection, available records, calculations and past operational performance, Pines Lake Dam, a high hazard potential structure, is judged to be in fairly good overall condition. However, the dam's spillway is considered inadequate since 50 percent of the Probable Maximum Flood (PMF) would overtop the dam. To insure adequacy of the structure, the following actions, as a minimum, are recommended:

a. The spillway's adequacy should be determined by a qualified professional consultant, engaged by the owner, using more sophisticated methods, procedures and studies initiated within one month and completed within six months from the date of approval of this report. Any remedial measures necessary to insure the adequacy of the spillway and to prevent overtopping should be initiated within calendar year 1979. In the interim, detailed emergency operation and evacuation plans and a warning system, should be promptly developed. Also, during periods of unusually heavy precipitation, around-the-clock surveillance should be provided.

b. Within one month from the date of approval of this report engineering studies and analyses should be initiated to assess the seepage conditions of the abutments and valley bottom, particularly the left (east) abutment area, and to investigate the effect of this seepage on the structural stability of the dam. Any remedial measures found necessary should be initiated within calendar year 1979.

NAPEN-D
Honorable Brendan T. Byrne

c. Within six months of the date of approval of this report, engineering studies and analyses should be initiated to develop remedial measures to repair the badly undermined stilling basin concrete weir located approximately 40 feet downstream from the dam. The recommended repairs should be initiated within calendar year 1979.

d. The following actions should be taken within the below listed times from the date of approval of this report:

(1) The eroded gully located along the downstream toe of the dam at the left abutment area should be properly filled and stabilized within six months.

(2) A new outlet pipe or other system to drawdown the lake should be installed within one year.

(3) Spalled concrete and gunite on the dam, roadway, and supporting cantilever sections should be repaired periodically to prevent further deterioration.

A copy of the report is being furnished to Mr. Dirk C. Hofman, New Jersey Department of Environmental Protection, the designated State Office contact for this program. Within five days of the date of this letter, a copy will also be sent to Congressman Robert A. Roe of the Eighth District. Under the provisions of the Freedom of Information Act, the inspection report will be subject to release by this office, upon request, five days after the date of this letter.

Additional copies of this report may be obtained from the National Technical Information Services (NTIS), Springfield, Virginia, 22161 at a reasonable cost. Please allow four to six weeks from the date of this letter for NTIS to have copies of the report available.

An important aspect of the Dam Safety Program will be the implementation of the recommendations made as a result of the inspection. We accordingly request that we be advised of proposed actions taken by the State to implement our recommendations.

Sincerely yours,

JOEL T. CALLAHAN
Lieutenant Colonel, Corps of Engineers
Acting District Engineer

1 Incl
As stated

Cy furn:
Mr. Dirk C. Hofman, P.E., Deputy Director
Division of Water Resources
N. J. Dept. of Environmental Protection
P.O. Box 2809
Trenton, NJ 08625

PINES LAKE DAM
Inventory Number: 00250

Corps of Engineers Assessment of General Conditions

This dam was inspected on 19 June 1978 by Michael Baker, Jr., Consulting Engineers under contract to the U. S. Army Engineer District, Philadelphia, in accordance with the National Dam Inspection Act, P.L. 92-367.

The Pines Lake Dam, a high hazard potential structure, is judged to be in fairly good overall condition. However, the dam's spillway is considered inadequate since 50 percent of the Probable Maximum Flood (PMF) would overtop the dam. To insure adequacy of the structure, the following actions, as a minimum are recommended:

a. The spillway's adequacy should be determined by a qualified professional consultant, engaged by the owner, using more sophisicated methods, procedures and studies initiated within one month and completed within six months from the date of approval of this report. Any remedial measures necessary to insure the adequacy of the spillway and to prevent overtopping should be initiated within calendar year 1979. In the interim, detailed emergency operation & evacuation plans and a warning system, should be promptly developed. Also, during periods of unusually heavy precipitation around-the-clock surveillance should be provided.

b. Within one month from the date of approval of this report engineering studies and analysis should be initiated to assess the seepage conditions of the abutments and valley bottom, particularly the left (east) abutment area, and to investigate the effect of this seepage on the structural stability of the dam. Any remedial measures found necessary should be initiated within calendar year 1979.

c. Within six months of the date of approval of this report engineering studies and analysis should be initiated to develop remedial measures to repair the badly undermined stilling basin concrete weir located approximately 40 feet downstream from the dam. The recommended repairs should be initiated within calendar year 1979.

d. The following actions should be taken within the below listed times from the date of approval of this report:

The eroded gully located along the downstream toe of the dam at the left abutment area should be properly filled and stabilized within six months.

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Corps of Engineers Assessment of General Conditions (Cont)

A new outlet pipe or other system to drawdown the lake should be installed within one year.

Spalled concrete and gunite on the dam, roadway, and supporting cantilever sections should be repaired periodically to prevent further deterioration.

APPROVED: Joel T. Callahan
JOEL T. CALLAHAN
Lieutenant Colonel, Corps of Engineers
Acting District Engineer

DATE: 19 September 1978

PHASE I REPORT
NATIONAL DAM SAFETY PROGRAM

Name of Dam - Pines Lake Dam, Passaic County, New Jersey
Stream - Haycock Brook
Date of Inspection - 19 June 1978

ASSESSMENT OF
GENERAL CONDITIONS

Pines Lake Dam is a concrete arch dam approximately 42 feet high and 127 feet long, owned and operated by the Pines Lake Association, Wayne, New Jersey.

The visual inspections and review of engineering data, made in June 1978, indicate no serious deficiencies requiring emergency attention. It is recommended, however, that the owner should immediately retain a qualified consultant to conduct further investigation on the seepage conditions of both abutments, particularly the east abutment area. The investigation should include an assessment of the effect of seepage on the structural stability of the dam and should develop necessary remedial measures. It is also recommended that the owner engage an engineering consultant to develop permanent remedial measures to repair the badly undermined stilling basin concrete weir and develop emergency operations procedures for the dam and reservoir, including emergency warning and evacuation plans for areas which will be affected in the event of a dam failure.

Hydraulic/hydrologic evaluations performed in accordance with established Corps of Engineers procedures for Phase I Inspection Reports revealed that the spillway will not pass the Probable Maximum Flood without overtopping the dam. Therefore, the owner should soon initiate an engineering study to evaluate the spillway capacity and to develop recommendations for remedial measures to reduce the overtopping potential of the dam.

MICHAEL BAKER, JR., INC.

Michael Baker, III, P.E.
Chairman of the Board and
Chief Executive Officer
Registration Number 13385

NAME OF DAM: PINES LAKE DAM

OVERALL VIEW OF DAM

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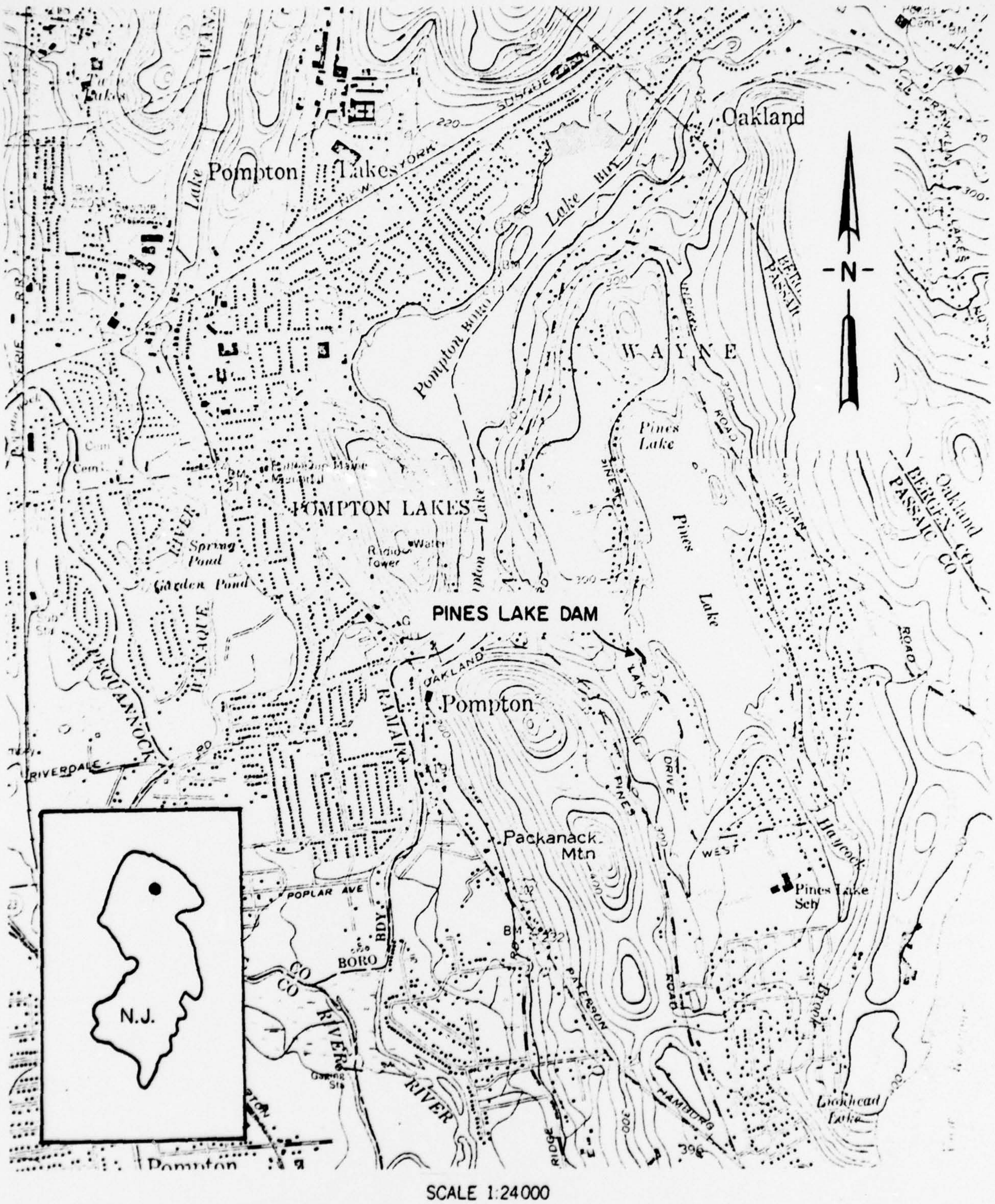

LOCATION PLAN
PINES LAKE DAM

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM
NAME OF DAM: PINES LAKE DAM, ID# NJ 00250

SECTION 1 - PROJECT INFORMATION

1.1 GENERAL

a. Authority - This report is authorized by the National Dam Inspection Act, Public Law 92-367, 92nd Congress, H.R. 15951 enacted 8 August 1972 and has been prepared in accordance with Contract No. DACW61-78-C-0141 between Michael Baker, Jr., Inc. and the U.S. Army Corps of Engineers, Philadelphia District.

b. Purpose of Inspection - The purpose of this inspection is to evaluate the general condition of Pines Lake Dam with respect to safety of the facility based upon available data and visual inspection.

1.2 DESCRIPTION OF PROJECT

a. Description of Dam and Appurtenances - The Pines Lake Dam consists of a gravity arch approximately 42 feet high and 127 feet long. Seepage control is provided by concrete abutments keyed into the bedrock, interbedded shale and sandstone. The intake structure (and outlet works) consists of a 36 inch diameter sluice gate which is no longer operational. The discharge is controlled by a variable height dropped-crest section. The spillway is a broad crested weir.

b. Location - Pines Lake Dam is located on the Haycock Brook approximately six miles east of the town of Pompton Lakes, Passaic County, New Jersey.

c. Size Classification - The maximum height of the dam is 42 feet. The reservoir volume to the top of dam is 6400 acre-feet. Therefore, the dam is in the "Intermediate" size category as defined by the "Recommended Guidelines for Safety Inspection of Dams."

d. Hazard Classification - Due to the proximity of the town of Pompton Lake, New Jersey, with a population of about 11,000, many lives could be lost in the event of failure of the dam. Therefore, this dam is considered in the "High" hazard category as defined by the "Recommended Guidelines for Safety Inspection of Dams."

e. Ownership - The dam is owned by the Pines Lake Association, Wayne, New Jersey.

f. Purpose of Dam - The dam is used for recreational purposes.

g. Design and Construction History - The existing facility was designed for the owner by William I. Whitmore Engineerings of Peterson, New Jersey and F. W. Schwiers, Jr., Consulting Engineer of New York, New York. The dam was built by F. W. Schwiers, Jr., Inc., New York, beginning in May 1927. Construction was completed in October 1927.

h. Normal Operational Procedures - No formal operating procedures are followed for the dam. The Pines Lake Association is responsible for the operation and maintenance of the dam.

1.3 PERTINENT DATA

a. Drainage Area - The drainage area of the Pines Lake Dam is 3.7 square miles.

b. Discharge at Damsite - The maximum flow at the damsite through the dropped-crest section is not known.

c. Elevation [feet above Mean Sea Level (M.S.L.)] -

Top of Dam - 261.0
Maximum Pool (Design Discharge) - 258.3 (144 c.f.s. at one foot head)
Recreation Pool - 257.30
Streambed at Centerline of Dam - 219.0
Maximum Tailwater - Not available

d. Reservoir (miles) -

Length of Maximum Pool - 1.1 approximately
Length of Recreation Pool - one

e. Storage (acre-feet) -

At Spillway Crest (El. 257.30) - 5880 (from Appendix C)
Top of Dam (El. 261.0) - 6400 (estimated from U.S.G.S. 7.5 minute topographic quadrangle)

f. Reservoir Surface (acres) -

Top of Dam - 146.0
Spillway Crest - 134.0
Maximum Pool - Not available
Normal Pool - 134.0

g. Dam -

Type - Concrete arch
Length - 127 feet
Height - 42 feet
Top Width - 8.0 feet
Side Slopes - Upstream - Vertical
Downstream - Approximately one horizontal to five vertical (1:5)
Impervious Core - Not applicable
Cutoff - Not applicable

h. Diversion and Regulating Tunnel - None

i. Spillway -

Type - Broad crested weir
Length of Weir - 50 feet (five weirs with 10 feet widths)
Crest Elevation - 257.30 feet (M.S.L.)
Gates - None
Downstream Channel - A plunge pool extends approximately 40 feet downstream from the toe of the dam. Average width is 40 feet, and the average depth of the pool was found to be 2.5 feet.

j. Regulating Outlets - None

SECTION 2 - ENGINEERING DATA

2.1 DESIGN

Design information reviewed included:

1) Drawings: "Plans for a Concrete Arch Dam at Pompton, New Jersey" by W. L. Whitmore and F. W. Schwiers, Jr., 4 March 1927; "Pines Lake Corporation - Design for Roadway on Top of Dam," by F. W. Schweirs, Jr., 14 July 1927, and its revised drawing dated 16 July 1927. Those three drawings, showing plan, elevation and section of the dam, are contained in the microfiche files of New Jersey Department of Environmental Protection (N.J.D.E.P.)

2) Notes, correspondence, original application for the dam, some monthly construction progress reports, and occasional inspection reports from microfiche files of N.J.D.E.P. Only documents available from 1927 to 1968 in the microfiche files are available for review.

2.2 CONSTRUCTION

The dam was constructed between May and October 1927 by F. W. Schwiers, Jr., Co., Engineers and Contractors of New York City. Construction progress reports are on file at the N.J.D.E.P. No "as built" drawings of the dam were available for review. Observations during the field inspection indicate that visible portions of the dam were generally constructed in accordance with the design drawings prepared by W. L. Whitmore and F. W. Schwiers in March 1927.

Chemical (AM-9) grouting of the downstream right abutment rock was done by Sprague and Henwood of Scranton, Pennsylvania during November and December of 1961. The stilling basin concrete weir, 40 feet downstream from the dam, was reconstructed in 1962. A sewer line was constructed across the valley on support bents 80 feet downstream from the dam in 1965. Areas of spalled concrete on the upstream and downstream faces of dam were treated with gunite, and the outlet conduit was plugged with concrete in 1967.

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2.3 OPERATION

The Pines Lake Association is responsible for the operation and maintenance of the Pines Lake Dam which includes maintaining the smaller downstream concrete weir. The concrete weir is used to contain a small plunge pool to prevent scour at the downstream toe of Pines Lake Dam.

2.4 EVALUATION

The readily available information in the microfiche files of the N.J.D.E.P.; an inspection report dated October 8, 1976 prepared by Thor Engineers of Newark, New Jersey and New York, New York; plus observations made during the field inspection are considered adequate for the purposes of this Phase I Inspection Report.

SECTION 3 - VISUAL INSPECTION

3.1 FINDINGS

a. General - The dam and its appurtenant structures were found to be in fairly good overall condition at the time of inspection. Most of the problems noted during the visual inspection are considered minor and do not require immediate remedial treatment. The problems requiring immediate attention are the areas of seepage at both abutment areas, particularly the east abutment area. The seepage quantity in the east abutment area appeared to have increased since the inspection performed in October 1976 by the owner's consultant, Thor Engineers. It is impossible to assess from the type of visual inspection performed for this Phase I Inspection Report; the extent to which the observed seepage will affect the structural integrity of the dam. For this reason, it is recommended that the owner immediately retain a qualified consultant to evaluate and rectify this condition. The other problem, which requires immediate attention, is the erosion condition in front of the dam at the left abutment area.

Other significant observations made during visual inspection of Pines Lake Dam by Michael Baker, Jr., Inc. are presented briefly in the following paragraphs. The complete visual inspection check list is given in Appendix A.

b. Dam and Appurtenant Structure - The visual inspection of Pines Lake Dam performed on 19 June 1978 revealed that the dam structure was in fairly good condition. The downstream face of the dam and the crest showed evidence of gunite repair. The gunite was in fair condition with a four feet long crack adjacent to the right rock abutment approximately six feet above water level. There was evidence of a ripple effect in the gunite to the right of the overflow spillway. This condition may be due to the presence of water between the gunite and the original concrete.

No gunite was noted to be present beneath the overflow spillway. There was evidence of minor spalling in the area.

There was also spalling of the gunite in the area adjacent to the left abutment just above water level.

The concrete roadway and supporting concrete cantilever sections were badly spalled, and evidence of calcite was present throughout.

A two feet high concrete weir located approximately 40 feet downstream of the dam, that was built in 1962, has been undermined. Consequently, water was flowing under the weir at the time of inspection. The function of the weir is to create a plunge pool to act as an energy dissipator.

Seepage was observed near both rock abutments. It is believed that the seepage has been present for quite some time. The seepage at the right abutment was minimal flowing at approximately one g.p.m. This rock abutment had been grouted in 1961. The seepage at the left abutment was more significant with an approximate rate of about seven g.p.m., which was an increase over the estimated three g.p.m. observed in October 1976. The seepage was present at higher ground elevations, indicating flow from the reservoir as opposed to normal groundwater flow. The seepage was generally confined to the area just downstream of the dam.

An erosion gully was present along the downstream toe of the dam at the left abutment area. If some remedial work is not done, this erosion condition will worsen.

c. <u>Reservoir Area</u> - Most of the reservoir slopes are relatively flat and well-forested; some are grass covered. The reservoir slopes are predominately glacial soil, and there is evidence of some localized erosion.

d. <u>Downstream Channel</u> - The channel downstream from the stilling basin, which has a gradient of approximately two percent, is the natural stream channel of Haycock Brook. The stream channel is approximately 18 to 20 feet wide, and the bottom of the channel is mostly covered with small to medium size rock fragments. A few, large, fallen trees were partially blocking the stream channel in a sharp bend 700 feet downstream from the dam. The downstream channel area slopes are moderately sloping except for a conglomeritic sandstone cliff about 30 feet high on the northwest side of the stream, 200 to 300 feet downstream from the dam. The slopes are heavily forested with mature stands of trees.

3.2 EVALUATION

The dam shows no evidence of slope instability or structural deficiencies. There is no evidence of any major scour conditions at the base of the dam; however, if the concrete weir which contains the plunge pool continues to deteriorate and is not capable of containing a sufficient amount of water, a scour problem will develop.

The seepage conditions on both abutments should be investigated further, especially in the left (east) abutment since the condition is progressively worsening.

SECTION 4 - OPERATIONAL PROCEDURES

4.1 PROCEDURES

Operational procedures are generally discussed in paragraphs 1.2.h and 2.3.

There is no formal written procedure for emergency downstream evacuation in the event of impending catastrophe; however, if conditions warrant such measures, the local authorities would be notified and appropriate evacuation measures would be taken. It is recommended that a formal emergency procedure be prepared and prominently displayed, and furnished to all appropriate personnel.

Rapid emergency drawdown is impossible as there are no functional outlet works. Water must be pumped or siphoned over the dam to drawdown the lake. Consideration should be given to developing emergency operating procedures.

4.2 MAINTENANCE OF DAM

The Pines Lake Association is responsible for the maintenance of the dam. At the time of this report preparation, the association did not have a regular maintenance schedule for Pines Lake Dam.

4.3 MAINTENANCE OF OPERATING FACILITIES

The Pines Lake Association is responsible for maintaining the dam and its facilities. Very little maintenance is required, as there is no operating equipment which must be maintained. Periodically, the association will hire a consultant to inspect the dam.

4.4 WARNING SYSTEMS

At present, there are no warning systems in operation for this dam.

4.5 EVALUATION

Maintenance of the operating facilities and the dam in general is adequate. There are no items which require constant maintenance, and the Pines Lake Association conducts periodic inspections.

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SECTION 5 - HYDRAULIC/HYDROLOGIC

5.1 EVALUATION OF FEATURES

a. Design Data - Very limited data is available in the microfiche files of N.J.D.E.P.

b. Experience Data - Experience data was not available for review and evaluation.

c. Visual Observations - Observation during the field inspection of the dam indicated that the spillway functions in the manner for which it was designed. Minor spalling of concrete was observed on the central weir. A small amount of calcite deposit on the concrete was noted on the four remaining weirs. The two feet high stilling basin weir located approximately 40 feet downstream from the dam was observed to be undermined along most of its length.

d. Overtopping Potential - The Pines Lake Dam is classified as a "High" hazard-"Intermediate" size dam requiring evaluation for a spillway design flood equal to the Probable Maximum Flood (P.M.F.). The spillway is composed of a series of five rectangular 10 feet wide notches separated by piers and symetical about the centerline of the arch dam. The central notch, at El. 257.3, carries normal flow while the others (two at El. 257.5 and two at El. 257.8) assist during floods. Directly above the notches and on top of the piers is a roadway. The low chord is at El. 259.5 and the top at road at El. 261.0. The spillway rating curve was developed by methods presented in the Handbook of Hydraulics by King and Brater using design drawings and field measurements to obtain spillway dimensions. The maximum spillway capacity of 800 c.f.s. is developed when the pool level is at top of dam El. 261.0.

The hydrologic analysis to determine the P.M.F. was developed by using methods presented in Design of Small Dams, a U.S. Bureau of Reclamation publication; EM-1110-2-163, a U.S. Army Corps of Engineers publication; and with the aid of the HEC-1 Flood Hydrograph Package, a U.S. Army Corps of Engineers Computer Program. The development of the P.M.F. for Pines Lake Dam involves the evaluation of both controlled and uncontrolled drainage areas to account for regulation by the Lionshead and Point View dams, both located upstream of Pines Lake Dam.

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The evaluation of the uncontrolled drainage area (1.66 square miles) was first completed assuming no flow from the upstream reservoirs. Using the uncontrolled area and the procedures outlined above, a peak discharge of 6830 c.f.s. was developed for the P.M.F. This flow was routed through the reservoir and found to overtop the dam by 2.5 feet. The spillway was therefore assessed as inadequate. The above evaluation is based only upon the uncontrolled drainage. Therefore if the effects of the upstream dams were taken into account, the dam would be overtopped by a greater amount. An in-depth study of a combination of all three dams was not completed for this study. However, based upon the large storage volume and retention time (4.3 hours from N.J.D.E.P. microfiche) of the Point View Dam reservoir, a large increase above the uncontrolled P.M.F. peak discharge is not expected. The uncontrolled peak discharge was therefore used for this analysis.

In order to assess the overtopping potential of Pines Lake Dam, a flood discharge equal to approximately one-half P.M.F. was routed through the dam. The one-half P.M.F. value crested at 0.1 foot above the top of dam. Based upon these routings, it is estimated that the dam will pass a flood flow equal to approximately 49 percent of the P.M.F. value from the uncontrolled drainage area or approximately 45 percent of the P.M.F. value of the total drainage area.

e. <u>Emergency Drawdown of Lake</u> - Rapid emergency drawdown of the lake is impossible. There are no functional outlet works, since the outlet conduit was plugged with concrete in 1967. The lake can only be drawn down by pumping or siphoning water over the dam.

SECTION 6 - STRUCTURAL STABILITY

6.1 EVALUATION OF STRUCTURAL STABILITY

a. Visual Observations - No structural inadequacies, in the dam itself, were noted at the time of inspection. There was no evidence of slope instability, structural cracking or of any serious scour condition present at the dam. There was, however, an erosion condition in front of the dam at the left abutment area that requires immediate attention.

Seepage was present at both rock abutments. Grouting was performed on the right (west) abutment in 1961, and no significant increase in seepage has been noticed since.

The left abutment had significant seepage present. Based on previous inspection reports and observations, this condition is becoming worse. It will be necessary to investigate the source and seriousness of this condition, and take appropriate remedial action.

The small concrete weir located downstream of the dam was observed to have been undermined. This condition should be corrected; such that, a sufficient depth of water is maintained at the toe of the dam to prevent a serious scour condition which would affect the stability of the dam foundation.

b. Design and Construction Data - Very little design and construction data were available to evaluate the structural stability of the dam. The past performance of the dam and observations made during the field inspection suggest that the dam should satisfy the structural requirements outlined in Section 4.4 of the "Recommended Guidelines for Safety Inspection of Dams" (attached to this report as Appendix D). However, considering the seepage condition on the abutment areas and the lack of design data for stability assessment, the structural stability should be further evaluated immediately by a qualified consultant retained by the owner.

c. Operating Records - Nothing in the readily available operating records suggests structural inadequacy of the dam.

d. Post-Construction Changes - Four major post-construction activities have been undertaken for the dam:

1) Grouting of the right abutment in 1961.

2) Reconstruction of the downstream stilling basin concrete weir in 1962.

3) Application of gunite on the downstream face of the dam and on the concrete abutments in 1967.

4) Dismantling the sluice gate and plugging the outlet pipe in 1967.

These post-construction activities, excluding item 4), were maintenance procedures and enhanced the structural stability of the dam. The dismantling of the sluice gate and the plugging of the outlet pipe with concrete (noted in Plate 1) have virtually eliminated any means of drawdown of the lake, particularly in case of emergency. This may adversely affect the structural stability of the dam in case of high flood or emergency conditions.

e. Seismic Stability - Pines Lake Dam is located in Seismic Zone 1 according to the "Seismic Zone Map of the Contiguous United States" (Figure 1, page D-30) from the "Recommended Guidelines for Safety Inspection of Dams." This is a zone of low seismic activity. Experience indicates that dams in Seismic Zone 1 will have adequate stability under seismic loading conditions, if they have adequate stability under static loading conditions. This point should be addressed further by the consultant who evaluates structural stability in conjunction with seepage conditions of the dam as previously mentioned in paragraphs 6.1.a. and 6.1.b.

SECTION 7 - ASSESSMENT, RECOMMENDATIONS/REMEDIAL MEASURES

7.1 DAM ASSESSMENT

a. Safety - There are no detrimental findings, as a result of conditions observed during the visual inspection on 19 June 1978, by which an inadequate assessment of the structural stability of Pines Lake Dam can be rendered; provided the dam is not overtopped by flood waters. The hydraulic/hydrologic analysis performed in accordance with Corps of Engineers procedures for Phase I Inspection Reports (paragraph 5.1) has indicated that the spillway of the Pines Lake Dam is inadequate to pass the P.M.F. without the dam being overtopped by about 2.5 feet of water.

b. Adequacy of Information - The readily available information and the observations made during the field inspection of the dam are considered sufficient for purposes of this Phase I Inspection Report.

c. Urgency - The dam does not require urgent remedial work.

d. Necessity for Further Investigation - The owner should immediately retain a qualified consultant to assess the seepage conditions of the abutments and valley bottom, particularly the left (east) abutment area, and to investigate the effect of this seepage on the structural stability of the dam. This consultant should also perform an in-depth hydraulic and hydrological analysis to determine the feasibility of increasing the capacity of the spillway in order to meet acceptable spillway criteria. A study, with subsequent necessary repair work, should also be made without undue delay for the badly undermined concrete weir of the downstream stilling basin.

7.2 RECOMMENDATIONS/REMEDIAL MEASURES

As described in paragraph 7.1.d., the dam inspection revealed certain items requiring immediate attention. That is, the owner should immediately retain a qualified consultant to:

1) Assess the seepage conditions of the abutments and valley bottom, particularly the left (east) abutment area.

2) Investigate the effect of seepage on the structural stability of the dam.

3) Develop plans and specifications for remedial work as necessary.

Stability and seepage evaluations of the dam should be done, where applicable, in accordance with procedures and criteria outlined in Section 4.4 of the "Recommended Guidelines for Safety Inspection of Dams." A copy of Section 4.4 is included in this report as Appendix D for reference purposes.

In accordance with the established Corps of Engineers procedures for Phase I Inspection Reports, the spillway of Pines Lake Dam has been determined to be insufficient to pass the P.M.F. without the dam being overtopped. Consequently, the consultant retained by the owner should soon conduct an in-depth engineering study to evaluate the spillway capacity and to develop recommendations for remedial measures to reduce the overtopping potential of the dam.

The dam inspection revealed several other items which also require attention. They are listed as follows:

1) It is recommended that a formal emergency procedure be developed very soon and prominently displayed and furnished to all operating personnel. This should include:

   a) How to operate the dam and reservoir during an emergency.

   b) Procedures for rapid drawdown of the reservoir under emergency conditions.

   c) Who to notify, including public officials, in case evacuation from the downstream area is necessary.

   d) The owner should assist public officials in developing an emergency evacuation plan for areas which will be affected in the event of a dam failure.

2) Study should be made soon and remedial measures should be developed to repair the badly undermined stilling basin concrete weir located approximately 40 feet downstream from the dam.

3) The eroded gully located along the downstream toe of the dam at the left abutment area should be properly filled and stabilized soon.

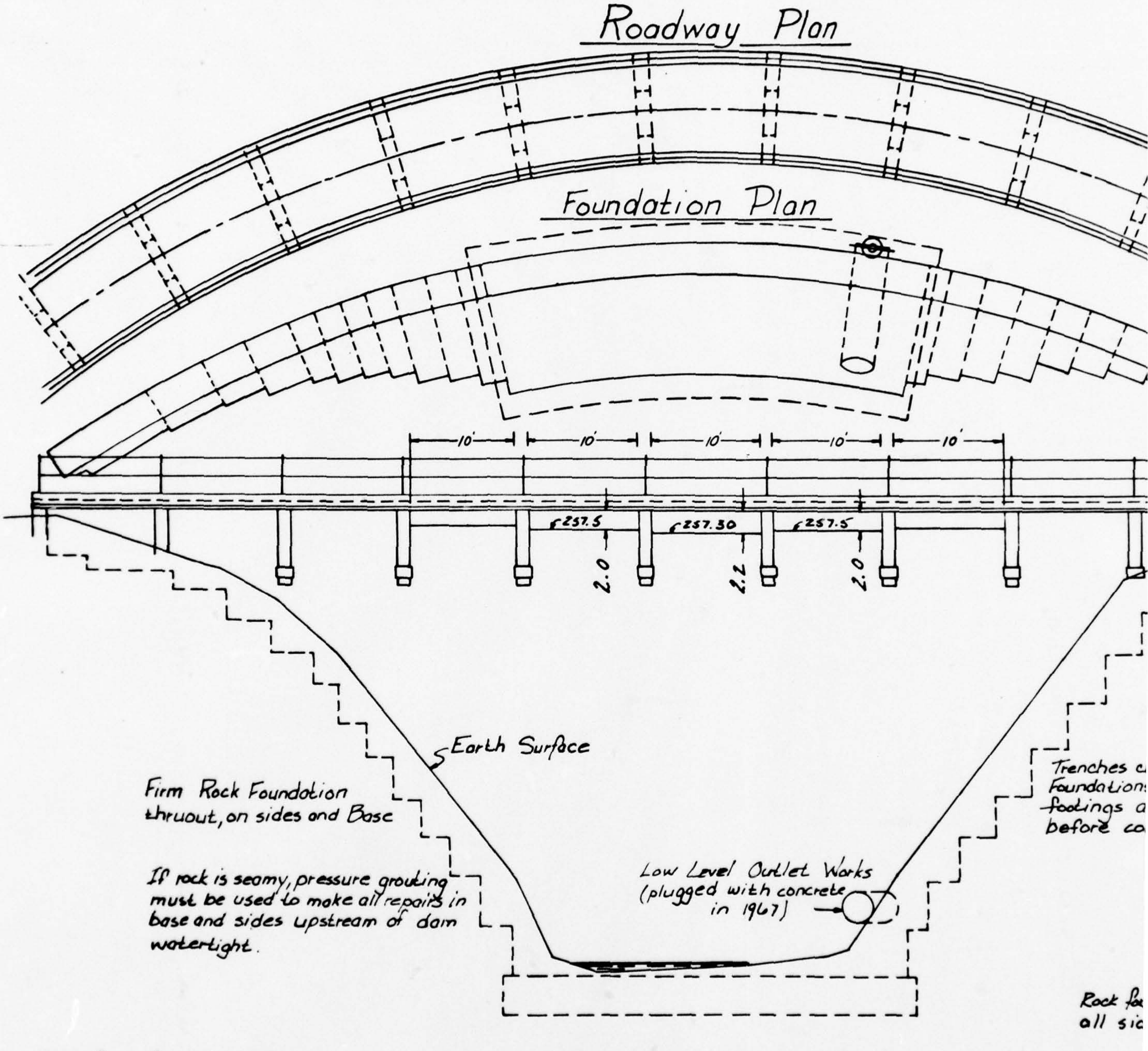
4) A new outlet pipe or other system to drawdown the lake should be installed in the near future.

5) Spalled concrete and gunite on the dam, roadway, and supporting cantilever sections should be repaired periodically to prevent further deterioration.

PLATES

NOTE: NO ORIGINAL DRAWINGS OF REPRODUCIBLE QUALITY WERE AVAILABLE FOR INCLUSION IN THIS REPORT. IF MICROFICHE COPIES OF THE ORIGINAL DRAWINGS ARE DESIRED, PLEASE CONTACT:

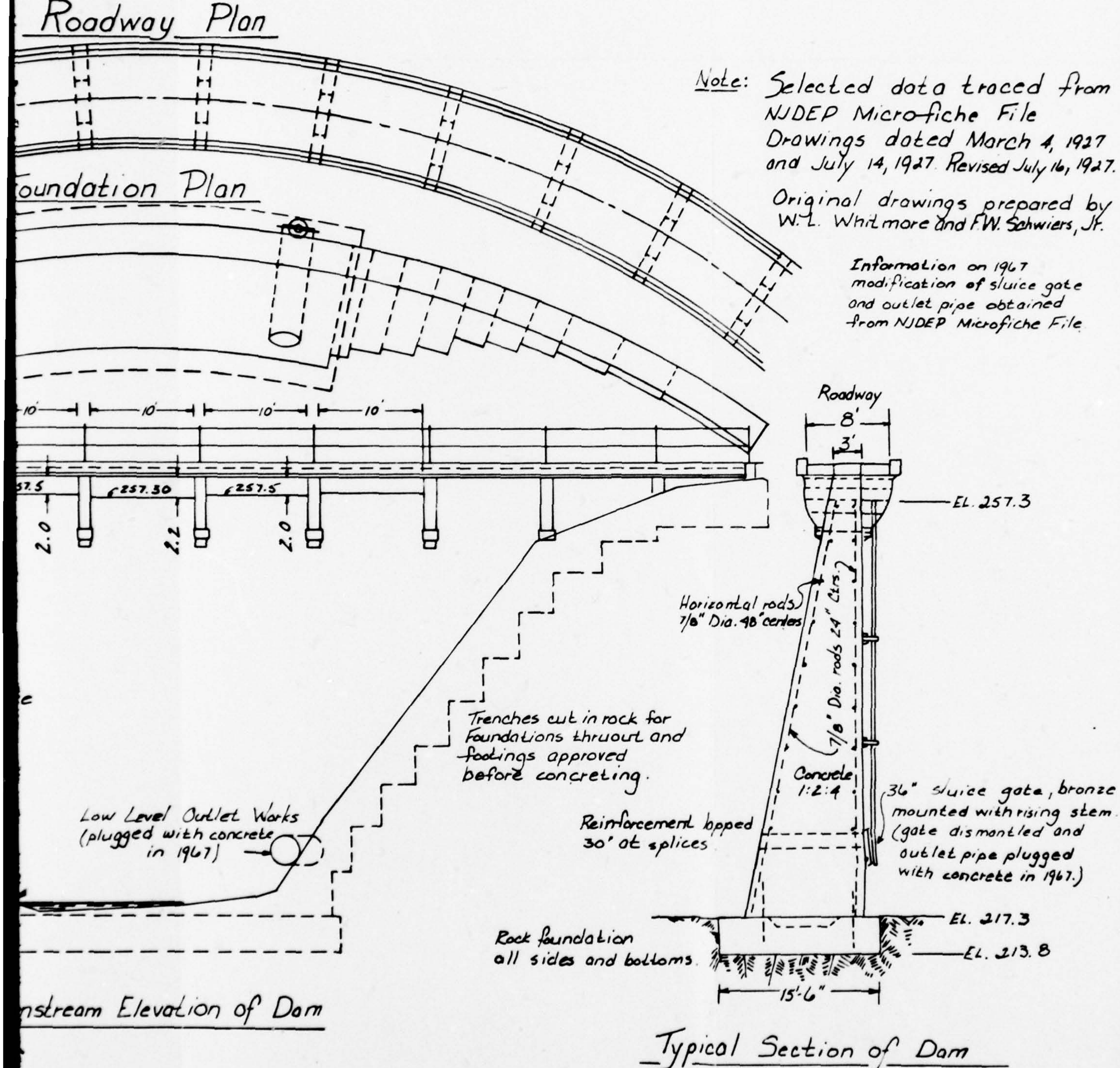
New Jersey Department of Environmental Protection
Division of Water Resources
Bureau of Water Control
P.O. Box 2809
Trenton, New Jersey 08625



Generalized Plans, Elevation and Typical Sc

Pines Lake Dam

Tracings made by
Michael Baker, Jr., Inc.
July 1978



Plans, Elevation and Typical Section of
Pines Lake Dam

PLATE 1

2

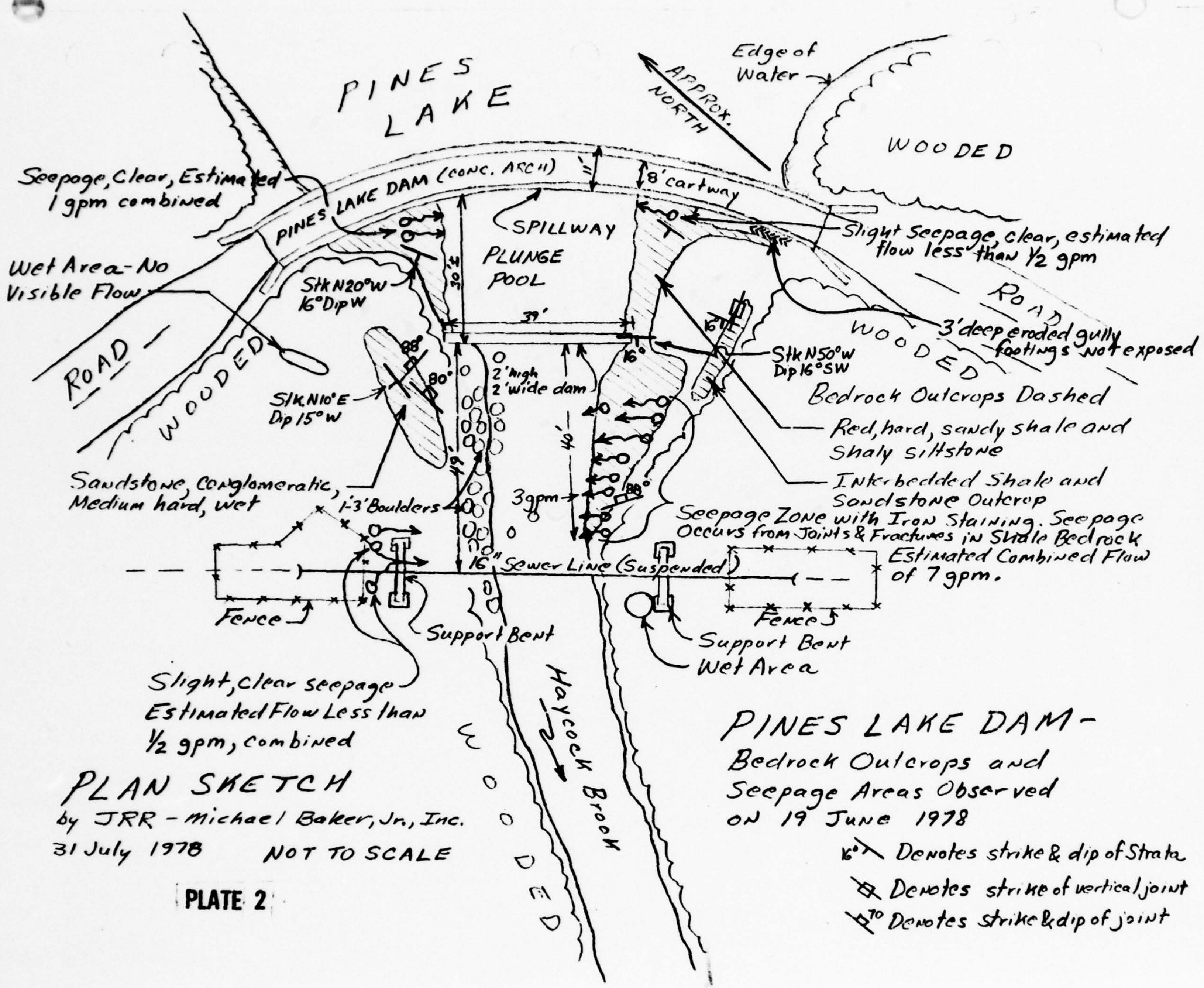

PINES LAKE
APPROX. NORTH
Edge of Water
WOODED
Seepage, Clear, Estimated 1 gpm combined
PINES LAKE DAM (CONC. ARCH)
11'
8' cartway
SPILLWAY
PLUNGE POOL
Slight Seepage, clear, estimated flow less than 1/2 gpm
Wet Area - No Visible Flow
Stk N20°W 16° Dip W
30.4'
39'
ROAD
WOODED
3' deep eroded gully footings not exposed
Stk N50°W Dip 16° SW
Bedrock Outcrops Dashed
Red, hard, sandy shale and Shaly siltstone
Interbedded Shale and Sandstone Outcrop
Stk N10°E Dip 15°W
2' high 2' wide dam
Sandstone, Conglomeratic, Medium hard, wet
1-3' Boulders
3gpm
Seepage Zone with Iron Staining. Seepage Occurs from Joints & Fractures in Shale Bedrock Estimated Combined Flow of 7 gpm.
16" Sewer Line (Suspended)
Fence
Support Bent
Wet Area
Slight, clear seepage Estimated Flow Less than 1/2 gpm, combined
Haycock Brook
PINES LAKE DAM - Bedrock Outcrops and Seepage Areas Observed on 19 June 1978
PLAN SKETCH by JRR - Michael Baker, Jr., Inc. 31 July 1978 NOT TO SCALE
Denotes strike & dip of Strata
Denotes strike of vertical joint
Denotes strike & dip of joint

PLATE 2

PHOTOGRAPHS

DETAILED PHOTOGRAPH DESCRIPTIONS

Overall View of Dam - View of Overflow Spillway and Roadway With Supports (Photograph Is Looking West From Left Abutment.) - 19 June 1978

Photo 1 - View of Roadway Above Dam Looking East From Right Abutment Area - 19 June 1978

Photo 2 - View of Overflow Spillway and Roadway Supports (Picture Was Taken Upstream From Dam Looking South. Note Spalled Concrete Roadway Slab.) - 19 June 1978

Photo 3 - Undermined Area of Small Dam Which Retains Plunge Pool -19 June 1978

Photo 4 - Seepage From Dipping Shale Bedrock Outcrop Located Downstream From Left Abutment - 19 June 1978

Photo 5 - Close-Up of Seepage From Shale Bedrock Approximately 45 Feet Downstream From Left Abutment (Note Staining of the Bedrock Indicating Continuous Flow.) - 19 June 1978

Photo 6 - View of Left Abutment Concrete-Shale Bedrock Junction at Front Face of the Dam With Slight Seepage - 19 June 1978

Photo 7 - View of Right Abutment Area at the Concrete-Shale Bedrock Junction at Front Face of the Dam (Note Crack in Gunite Surface and Slight Seepage From Bedrock.) - 19 June 1978

Photo 8 - View of Spillway Showing spalling of Gunite Veneer - 19 June 1978

PHOTO 1

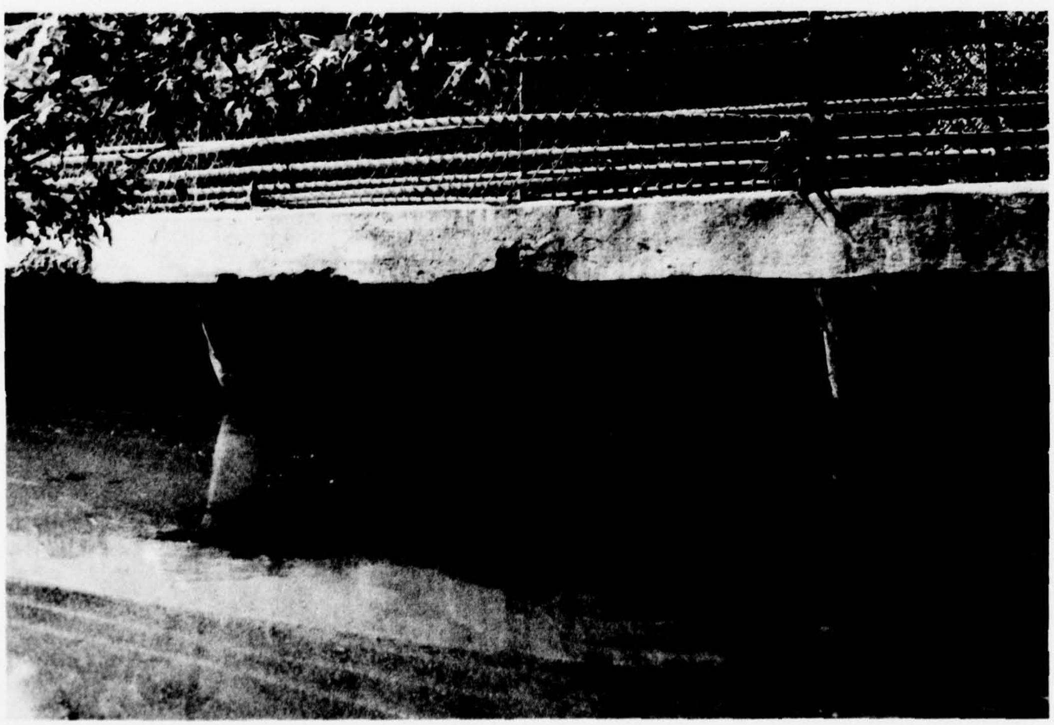


PHOTO 2

PHOTO 3

PHOTO 4

PHOTO 5

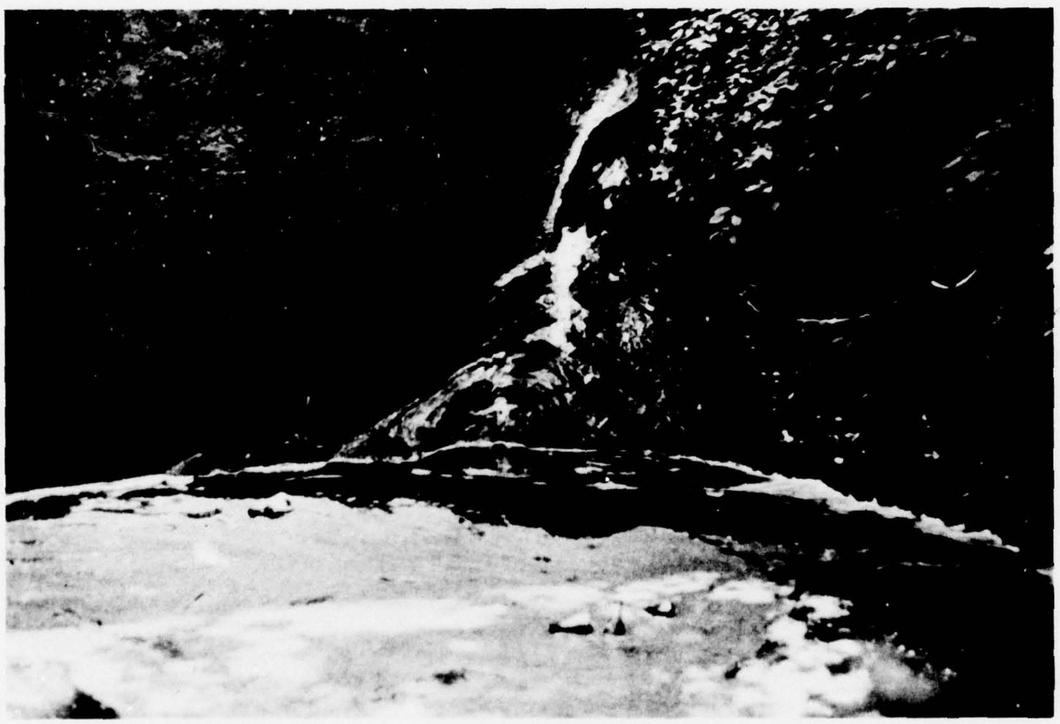


PHOTO 6

PHOTO 7

PHOTO 8

APPENDIX A

CHECK LIST - VISUAL INSPECTION

Check List
Visual Inspection
Phase 1

Name Dam Pines Lake Dam   County Passaic   State New Jersey   Coordinates Lat. 40° 59.5' Long. 74° 16.1'

Date Inspection 19 June 1978   Weather Warm, Sunny   Temperature 75°-80°F.

Note: The elevations indicated in this inspection report are based on the assumption that the top of the center spillway weir is at El. 257.30 feet as shown on revised plans dated 14 July 1927.

Pool Elevation at Time of Inspection 257.5' M.S.L.

Tailwater in Stilling Pond at Time of Inspection 222.4' M.S.L.

Tailwater downstream from stilling pond at time of Inspection 219.6' M.S.L.

Inspection Personnel:

MICHAEL BAKER, JR., INC.:

E. U. Gingrich
T. J. Dougan
J. R. Rapp

J. R. Rapp Recorder

Sheet 1

CONCRETE/MASONRY DAMS

REINFORCED CONCRETE ARCH DAM

PINES LAKE DAM

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SEEPAGE	See sketch with notes.	Sketch included as Plate 2 of Phase I Inspection Report.
STRUCTURE TO ABUTMENT/EMBANKMENT JUNCTIONS	Left abutment--small areas of spalled concrete. Right abutment--small amount of gunite has come loose from concrete.	
DRAINS	None	
WATER PASSAGES	None	
FOUNDATION	Foundations not exposed. Foundation consists of spread footings keyed into bedrock according to design drawings.	

CONCRETE/MASONRY DAMS

PINES LAKE DAM

REINFORCED CONCRETE ARCH DAM

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SURFACE CRACKS CONCRETE SURFACES	Roadway supports have small hairline cracks and calcite deposits.	
STRUCTURAL CRACKING	No evidence of structural cracking observed because original concrete has been covered with gunite. A four feet long crack was observed in gunite adjacent to right abutment six feet above tailwater.	Ripples in gunite surface right of spillway indicate loosening of gunite.
VERTICAL AND HORIZONTAL ALIGNMENT	No bulges or misalignments observed.	
MONOLITH JOINTS	None visible because of gunite surface coating.	
CONSTRUCTION JOINTS	None visible because of gunite surface coating.	

EMBANKMENT

PINES LAKE DAM

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SURFACE CRACKS	Not Applicable	
UNUSUAL MOVEMENT OR CRACKING AT OR BEYOND THE TOE	Not Applicable	
SLOUGHING OR EROSION OF EMBANKMENT AND ABUTMENT SLOPES	Not Applicable	
VERTICAL AND HORIZONTAL ALIGNMENT OF THE CREST	Not Applicable	
RIPRAP FAILURES	No riprap	

EMBANKMENT

PINES LAKE DAM

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
JUNCTION OF EMBANKMENT AND ABUTMENT, SPILLWAY AND DAM	Not Applicable	
ANY NOTICEABLE SEEPAGE	Not Applicable	
STAFF GAGE AND RECORDER	None present	
DRAINS	None present	

OUTLET WORKS

PINES LAKE DAM

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CRACKING AND SPALLING OF CONCRETE SURFACES IN OUTLET CONDUIT	No outlet conduit	
INTAKE STRUCTURE	None functioning	A 36 inch Ø sluice gate was designed and constructed in the dam. But according to information in microfiche files of N.J.D.E.P., the sluice gate was dismantled, and the outlet plugged with concrete in 1967.
OUTLET STRUCTURE	None	
OUTLET CHANNEL	None	
EMERGENCY GATE	None	

PINES LAKE DAM

UNGATED SPILLWAY

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONCRETE WEIRS (TOTAL OF FIVE)	Center weir, El. 257.3 M.S.L. - minor spalling of concrete. Two adjacent weirs, El. 257.5 M.S.L. - small amounts of scattered calcite deposits on concrete. Weirs nearest abutments, El. 257.8± M.S.L. - small amounts of scattered calcite deposits on concrete.	
APPROACH CHANNEL	None	
DISCHARGE CHANNEL	None, flow from dam drops into stilling basin. Flow from stilling basin occurs under two feet high stilling basin weir during low flow. Discharge is both over and under this weir as observed during higher flows on 14 June 1978. Flow enters natural stream channel below stilling basin weir.	
BRIDGE AND PIERS	Roadway with 8'-0" wide cartway is supported on Pines Lake Dam. Support piers and bottom of roadway deck are badly spalled, probably due to high water and to salt applied to road in winter.	
STILLING BASIN WEIR	Length--39 feet, height--two feet, width--two feet. The earth beneath the stilling basin weir has been eroded out for almost its entire length, such that a maximum of 10 inches of earth has been washed away at the middle of the weir. The concrete forming the weir was in good condition except for one crack in the middle about 1/16 inch wide which was probably caused by settlement.	The undermined portion of the stilling basin weir should be repaired.

GATED SPILLWAY

PINES LAKE DAM

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONCRETE SILL	Not Applicable	
APPROACH CHANNEL	Not Applicable	
DISCHARGE CHANNEL	Not Applicable	
BRIDGE AND PIERS	Not Applicable	
GATES AND OPERATION EQUIPMENT	None	

INSTRUMENTATION

PINES LAKE DAM

VISUAL EXAMINATION	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
MONUMENTATION/SURVEYS	Damsite surveyed either before, during or after construction. No survey monuments were noted during inspection.	
OBSERVATION WELLS	None	
WEIRS	None (except ungated spillway weirs).	
PIEZOMETERS	None	
OTHER	None	

RESERVOIR

PINES LAKE DAM

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SLOPES	Most of reservoir slopes are relatively flat and well-forested; some are grass covered. A few beach areas are present.	Glacial soils on reservoir slopes are quite stable from geotechnical and hydraulic viewpoints.
SEDIMENTATION	Two soundings from the center of the top of the dam indicated water depths of 28.2 and 29.7 feet for top of sediment elevations of 229.3 and 227.8, respectively. These measurements indicate a sediment depth of 10 to 12 feet upstream from the dam.	

DOWNSTREAM CHANNEL

PINES LAKE DAM

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONDITION (OBSTRUCTIONS, DEBRIS, ETC.)	The channel downstream from the stilling basin weir is the natural stream channel of Haycock Brook which has a gradient of two percent. The stream channel is approximately 18 to 20 feet wide and the bottom of the channel is mostly covered with small to medium size rock fragments. A few large fallen trees were partially blocking the stream channel in a sharp bend 700 feet downstream from dam.	
SLOPES	The downstream channel area slopes are moderately sloping except for a conglomerate sandstone cliff about 30 feet high on northwest side of stream, 200 to 300 feet downstream from Pines Lake Dam. The slopes have a heavy mature forest cover consisting of pine, sycamore and maple trees. No landslides were observed.	
APPROXIMATE NO. OF HOMES AND POPULATION	Reach 1 of Haycock Brook extends downstream approximately 700 feet from the dam through a narrow rock gorge to a horseshoe bend. Reach 2 extends approximately 1200 feet further downstream through a wider valley to a confluence with an unnamed stream. Reach 3 extends downstream another 1300 feet through an even wider valley to Pompton Lake. Reaches 1 and 2 are uninhabited. There are an estimated 20 homes and 100 people in Reach 3. (It is probable that Reaches 1, 2 and 3 were scoured by glacial melt water or the discharge from a glacial lake.) Pompton Lake is drained by the Romapo River. There are an estimated 100 homes and 500 people, plus other facilities in low lying areas along the reach of the Romapo River extending two miles downstream from Pompton Lake.	

APPENDIX B

CHECK LIST - ENGINEERING DATA

CHECK LIST
ENGINEERING DATA
DESIGN, CONSTRUCTION, OPERATION

PINES LAKE DAM

ITEM	REMARKS
PLAN OF DAM	Reference Drawings: "Plans for a Concrete Arch Dam at Pompton, New Jersey" by W. L. Whitmore and F. W. Schwiers, Jr., 4 March 1927; "Pines Lake Corporation - Design for Roadway on Top of Dam," by F. W. Schwiers, Jr., 14 July 1927, revised 16 July 1927. (Three drawings showing dam plan, elevation and section in microfiche files of N.J.D.E.P.) See Plate 1 of this report for plans traced from reference drawings.
REGIONAL VICINITY MAP	Section of U.S.G.S. Pompton Plains, New Jersey, 7.5 Minute Quadrangle included in this report as the Location Plan.
CONSTRUCTION HISTORY	Dam was constructed from May-October 1927 by F. W. Schwiers, Jr., Co., Engineers and Contractors, New York, New York. Chemical (AM-9) grouting of downstream right abutment rock was done by Sprague and Henwood, Scranton, Pennsylvania, in November-December 1961. The stilling dam 30 feet downstream from the main dam was re-constructed in 1962. A sewer line was constructed across valley on support bents 80 feet downstream from dam in 1965. Areas of spalled concrete on upstream and downstream faces of dam were treated with gunite and outlet conduit was plugged with concrete in 1967.
TYPICAL SECTIONS OF DAM	Reference Drawings and Plate 1 of this report.
HYDROLOGIC/HYDRAULIC DATA	Very limited data available in microfiche files of N.J.D.E.P.
OUTLETS - PLAN - DETAILS - CONSTRAINTS - DISCHARGE RATINGS	The 36 inch sluice gate and outlet conduit shown on Reference Drawings and Plate 1 of this report were constructed in 1927. A 1968 dam inspection report by Forrest Woodland, Jr., Consulting Engineer, indicates the outlet conduit was plugged with concrete in 1967. This report is in microfiche files of N.J.D.E.P. The dam had no outlet works at the time of inspection.
RAINFALL/RESERVOIR RECORDS	There is a stream gaging station located on Pompton Lake 0.8 mile downstream from Pines Lake Dam. Rainfall and gaging records are available there and probably also from the U.S.G.S. and the N.J.D.E.P.

ITEM	REMARKS
DESIGN REPORTS	No design calculations or reports are available.
GEOLOGY REPORTS	A three page report dated 3 January 1962 by L. E. Mark, Sprague and Henwood Field Engineer, on chemical grouting performed in the downstream right abutment in 1961 and an accompanying drawing dated 29 December 1961 are in the microfiche file of the N.J.D.E.P. Some geology information is given in the 8 October 1976 report by Thor Engineers which is included in Appendix C of this report.
DESIGN COMPUTATIONS HYDROLOGY & HYDRAULICS DAM STABILITY SEEPAGE STUDIES	The only design calculations available are those in the bottom left corner of the Reference Drawing dated 4 March 1927. These calculations, which are nearly illegible, appear to be related to arch reactions on the abutments and foundations.
MATERIALS INVESTIGATIONS BORING RECORDS LABORATORY FIELD	There are no records on material investigations, if any, at the time of dam design or construction. Some boring records from the 1961 grouting of the right abutment are available in the microfiche files of the N.J.D.E.P. Unconfined compression tests in 1965 gave strengths of 4523, 7226 and 6736 p.s.i. according to the 1968 dam inspection report by Forrest Woodland, Jr.
POST-CONSTRUCTION SURVEYS OF DAM	There is no information available on post-construction surveys of the dam.
BORROW SOURCES	The dam has no earthfill sections. No information is available on the sources of aggregate used in the dam concrete.

ITEM	REMARKS
MONITORING SYSTEMS	No monitoring systems are present.
MODIFICATIONS	Significant modifications to the dam were mentioned previously under the heading "CONSTRUCTION HISTORY."
HIGH POOL RECORDS	According to the 1968 dam inspection report by Forrest Woodland, Jr., 7.5 inches of rain fell during approximately 24 hours in May 1968 and the lake crested about two feet above spillway crest level. This appears to be the maximum pool of record.
POST-CONSTRUCTION ENGINEERING STUDIES AND REPORTS	Reports on dam inspections by New Jersey Department of Conservation personnel in 1929, 1942, 1960 and 1962 are available in the microfiche files of the N.J.D.E.P. An inspection made in 1956 by N. C. Harrison, Consulting Engineer, was mentioned in Hamson's report on his 1960 dam inspection which is in the microfiche files of N.J.D.E.P. along with the 1968 dam inspection report by Forrest Woodland, Jr. The 1976 dam inspection report by Thor Engineers is included in Appendix C of this report.
PRIOR ACCIDENTS OR FAILURE OF DAM DESCRIPTION REPORTS	No accidents or failures have been reported.
MAINTENANCE OPERATION RECORDS	No operations records are available; the dam has no operating equipment. Maintenance records are rather fragmentary. Some information on dam repairs is available in the microfiche files of the N.J.D.E.P. Additional maintenance information may be available from the present owner, though this is unlikely in view of the several changes in dam ownership over the years.

PINES LAKE DAM

ITEM	REMARKS
SPILLWAY PLAN	The reference drawings and Plate 1 of this report are the only available spillway drawings.
SECTIONS	
DETAILS	
OPERATING EQUIPMENT PLANS & DETAILS	There is no operating equipment.

CHECK LIST
HYDROLOGIC AND HYDRAULIC DATA
ENGINEERING DATA

DRAINAGE AREA CHARACTERISTICS: 3.7 square miles (2.04 square miles of watershed are controlled by Point View Dam and Lionshead Dam)

ELEVATION TOP NORMAL POOL (STORAGE CAPACITY): 257.3 (5586 acre-feet)

ELEVATION TOP FLOOD CONTROL POOL (STORAGE CAPACITY): Not available

ELEVATION MAXIMUM DESIGN POOL: 258.3

ELEVATION TOP DAM: 261.0

CREST: Broad crested weir spillway sections

a. Elevation 257.3 (center low flow section) 257.5-257.8 (four side sections)
b. Type Broad-crested weir
c. Width Approximately three feet
d. Length 50 feet, total--(five weir sections, each 10 feet wide)
e. Location Spillover Concrete weir
f. Number and Type of Gates No gates

OUTLET WORKS: Not operational

a. Type 36 inch sluice gate was dismantled in 1967 and outlet conduit was plugged with concrete
b. Location Low left side of dam
c. Entrance inverts Approximately El. 222
d. Exit inverts Approximately El. 222
e. Emergency draindown facilities Pumping or siphoning of water over dam

HYDROMETEOROLOGICAL GAGES: None

a. Type
b. Location
c. Records

MAXIMUM NON-DAMAGING DISCHARGE Not available

NAME OF DAM: PINES LAKE DAM

APPENDIX C

PINES LAKE DAM INSPECTION REPORT
OCTOBER 8, 1976

By: Thor Engineers
Newark, New Jersey - New York, New York

DAM INSPECTION REPORT

PINES LAKE

WAYNE, NEW JERSEY

October 8, 1976

Prepared for: Pines Lake Association

Thor Engineers
Newark, New Jersey - New York, New York

THOR ENGINEERS

17 ACADEMY STREET
NEWARK, NEW JERSEY 07102
201 - 624-3200

342 MADISON AVENUE
NEW YORK, NEW YORK 10017
212 - 682-7352

October 14, 1976

Pines Lake Association
C/O Mr. Thomas Mineo
16 Hawthorne Road
Wayne, New Jersey 07470

Re: Dam Inspection
Pines Lake Dam
Wayne, New Jersey

Gentlemen:

In accordance with your request and our proposal, we have performed a review of available information and a visual inspection of the subject concrete arch dam.

This report reflects our findings and conclusions and makes recommendations for further safety considerations. The considerations and procedures used for the inspection and report follow the "Recommended Guidelines for Safety Inspection of Dams" published by the Department of the Army, Office of the Chief of Engineers, Washington, D. C. The purpose of this approach is to satisfy the basic requirements of future legislation for dam safety. The program has been initiated and implemented with an inventory as outlined in The Dam Inspection Act, Public Law 92-367.

CLASSIFICATION OF DAM

The above mentioned guidelines categorizes the subject dam as follows:

SIZE CLASSIFICATION

Category	Storage (acre-feet)	Height (feet)
Intermediate	5,880	40

Hazard Potential Classification

Hazard potential must be considered as being high, due to the relatively large amount of downstream residential and commercial development.

PURPOSE AND SCOPE

The primary purpose of this investigation and report is to visually identify conditions that may pose present and/or future concern for dam stability and reservoir maintenance. The report shall consider what additional investigations and studies if any, are necessary to positively identify adverse conditions. The scope of this effort does not include the considerations of hydrologic and hydraulic features as they effect dam safety. Future implementation of the abovementioned legislation will require this work to complete an evaluation of safety.

INVESTIGATIONS

A thorough visual inspection of the dam was performed on August 21, 1976 by both a professional engineer and engineering geologist from this office. A subsequent site visit on October 1, 1976 allowed the collection of seepage data and downstream channel elevations. Photographs of apparent points of interest were taken and can be made available upon your request.

In addition to the above field exposure, the association's chronological file on dam related construction inspection and repair activity was reviewed in detail. The file was made available by you and assumed to reflect all previous activities.

FINDINGS

The existing dam is a reinforced concrete arch dam founded on the idigeneous rock, an interbedded shale and sandstone. Reservoir water spills over a dropped crest section and sheet flows down the downstream face of the concrete dam into a plunge pool. The reservoir shore line approaches the west abutment of the dam at a sharp angle. It curves around and runs a short distance south of the dam location in the area of the east abutment.

The dam structure shows evidence of gunite concrete repair over the downstream face, the crest and the roadway bridge structure over the crest. Spalling of concrete exists at the underside of the edge of the roadway deck and support members. The gunite veneer has spalled at various locations around the crest elevation and over the downstream face of the dam. Spalling appears to be more prevalent at the interface with the natural rock abutments and beneath the spillway flow area. Little or no veneer remains for the area subject to the spillway action.

There is no visual evidence of the low level outlet works as shown on the plans. The chronologic file indicates that the system had been concreted and hardware dismantled in 1967. An area of rock scour and apparent grout repair was noticed near the east abutment which is probably the remains of the outlet. Reports indicate that reservoir lowering is now accomplished with a siphon.

The plunge or energy dissipating pool at the base of the arch is littered with rock fragments, gravelly soil, timber and other organic debris. The water at the time of inspection was several feet deep. The elevation of the bottom of the pool was checked and found to average Elevation +218.2, approximately one (1) foot above the top of the arch footing elevation shown on the original construction drawings. A low concrete dam just downstream of the arch impounds the plunge pool. This small dam is undermined and water flows beneath it rather than over. The maximum flow is adjacent to the east bank of the channel. A pipe running through the small dam, apparently an outlet system, is totally clogged with rock fragment and soil and no flow was observed. The dam is literally a bridge as it is supported at only several points along its length.

Inspection of the natural rock and soil abutments from the arch dam contact to several hundred feet downstream indicated generally stable conditions as there was no evidence of recent overburden soil and rock sliding or slope failures. Large tree vegetation being relatively plumb enforces this conclusion. Seepage and at some points, leakage of water from the rock abutment and downstream banks does exist. Seepage and leakage is occurring at higher elevations and in greater quantities near the dam, indicating flow from the reservoir rather than normal land recharge of a groundwater system. Noticeable limonite staining exists at many points of discharge. Leakage is occurring at both east and west abutments, at and near the contact with the dam. The amount varies and appears to be flowing from fractures in the rock and not necessarily the contact surface. A quantity estimated at one (1) gallon per minute is flowing from the west abutment approximately eight (8)

feet above the channel profile and six (6) feet from the dam downstream face. The west abutment geology consists of a seam of silt and sand soil interbedded with the shale and sandstone rock, intersecting the channel section in the proximity of the referenced leak. This information was obtained during the fall of 1962 when exploratory borings were made as part of the program to cement and chemically grout the west abutment for a distance of approximately fifty (50) feet downstream to reduce leakage.

Leakage from the east abutment is further downstream from the point of dam contact. It is estimated at approximately three (3) gpm. This area appears to be recharged by the main reservoir and a shallow branch of the reservoir that runs south, parallel to the downstream profile for several hundred feet.

Based on field measurements, the interbedded sedimentary rock strata are striking approximately north-south and dipping west from 15 to 20 degrees.

CONCLUSIONS

Based on the above mentioned research and inspection, the dam and its foundation are in good condition with only cosmetic deterioration such as spalling of concrete around the edges and corners of the roadway deck and support members. Erosion and scour at the plunge pool and abutment have not progressed to a point of concern for dam stability. Seepage and leakage through the abutments and possibly through the channel bottom is occurring. Based on inspection and repair history, this flow is increasing with time. The west abutment leakage was apparently reduced to one (1) gpm in 1962 as a result of a

grouting program. The flow appears not to have increased. Leakage from the east abutment, appears to be increasing since little or no comment was made in past reports of what is now quite noticeable. The extent of limonite staining further enforces the theory of continual flow. This increased rate of flow is probably attributable to erosion and possible deterioration of the rock in the vicinity of joint, or crack systems that extend to the reservoir. The continuing flow and apparent consequent erosion being progressive will weaken the rock layer interfaces and may eventually create a hazard to dam stability. However, there is no need for immediate concern at this time. The attitude of the rock stratification indicates that the east abutment is structurally less stable in that sliding along the weaker interface of the rock layers could involve the natural abutment rock sliding into the downstream gorge. The west abutment would have to literally break off and rotate for failure to occur.

In summary, the following conclusions are offered:

1. The dam structure appears to be in good condition with only evidence of superficial concrete spalling.
2. The plunge pool bottom elevation indicates that no scour undermining of the dam foundation has occurred. However, dredging the pool and repairing the small dam is essential to maintain scour protection.
3. Leakage and seepage through the abutment and perhaps the base rock is occurring and can be expected to increase with time. The enlargement of the flow channels and water associated deterioration of rock near these channels may tend to weaken the abutments posing a stability problem.
4. The jointing and cracking of bedrock in the dam area is extensive and irregular with soft soil strata interbedding and, therefore, pose problems for effective grouting as a economic means of sealing.

RECOMMENDATIONS

There is no immediate need for concern for dam safety, however, as mentioned above, there exists a condition that could adversely effect the dam foundation. In the future we, therefore, recommend that a program be initiated to continually monitor the amount of leakage, seepage, and plunge pool bottom elevation. A typical program could include:

(1) Measure the seepage at points of leakage.

(2) Install observation wells to permit the monitoring and recording of data necessary to theoretically evaluate seepage conditions.

(3) Measure the flow over the dam crest and immediately downstream.

(4) Set up a survey bench mark so that elevation of bottom of plunge pool may be easily determined.

If the results of the above indicate a rapidly deteriorating condition, the following engineering activities are recommended to assist in evaluating viable remedial treatment for sealing the reservoir and protecting the abutments:

(1) Make subsurface and reservoir sub-bottm investigations to evaluate bedrock and sediment conditions.

(2) Make reservoir bottom soundings to ascertain reservoir contours in the proximity of dam.

Should excessive scour of the bottom of the plunge pool be realized, the area should be paved or filled with stone to reduce or eliminate scour.

The final evaluation of dam safety under a deteriorated foundation and the need for remedial treatment should include a detailed study of the hydrologic and hydraulic features of the dam and reservoir. These studies

will in any case, be required by future dam safety legislation especially for high hazard locations of which Pines Lake is one.

We trust our findings will satisfy your requirements and assist you with future plans regarding the safety of the dam and maintenance of Pines Lake.

Very truly yours,

THOR ENGINEERS

Thomas H. Otto

Thomas H. Otto, P.E.,
Partner

THO/tn

CC: File

APPENDIX D

EXCERPTS FROM
"RECOMMENDED GUIDELINES FOR SAFETY INSPECTION OF DAMS"

SECTION 4.4 - "STABILITY INVESTIGATIONS"
(Pages D-18 through D-27)

Reclamation and Soil Conservation Service. Many other agencies, educational facilities and private consultants can also provide expert advice. Regardless of where such expertise is based, the qualification of those individuals offering to provide it should be carefully examined and evaluated.

4.3.4. Freeboard Allowances. Guidelines on specific minimum freeboard allowances are not considered appropriate because of the many factors involved in such determinations. The investigator will have to assess the critical parameters for each project and develop its minimum requirement. Many projects are reasonably safe without freeboard allowance because they are designed for overtopping, or other factors minimize possible overtopping. Conversely, freeboard allowances of several feet may be necessary to provide a safe condition. Parameters that should be considered include the duration of high water levels in the reservoir during the design flood; the effective wind fetch and reservoir depth available to support wave generation; the probability of high wind speed occurring from a critical direction; the potential wave runup on the dam based on roughness and slope; and the ability of the dam to resist erosion from overtopping waves.

4.4. Stability Investigations. The Phase II stability investigations should be compatible with the guidelines of this paragraph.

4.4.1. Foundation and Material Investigations. The scope of the foundation and materials investigation should be limited to obtaining the information required to analyze the structural stability and to investigate any suspected condition which would adversely affect the safety of the dam. Such investigations may include borings to obtain concrete, embankment, soil foundation, and bedrock samples; testing specimens from these samples to determine the strength and elastic parameters of the materials, including the soft seams, joints, fault gouge and expansive clays or other critical materials in the foundation; determining the character of the bedrock including joints, bedding planes, fractures, faults, voids and caverns, and other geological irregularities; and installing instruments for determining movements, strains, suspected excessive internal seepage pressures, seepage gradients and uplift forces. Special investigations may be necessary where suspect rock types such as limestone, gypsum, salt, basalt, claystone, shales or others are involved in foundations or abutments in order to determine the extent of cavities, piping or other deficiencies in the rock foundation. A concrete core drilling program should be undertaken only when the existence of significant structural cracks is suspected or the general qualitative condition of the concrete is in doubt. The tests of materials will be necessary only where such data are lacking or are outdated.

4.4.2. Stability Assessment. Stability assessments should utilize in situ properties of the structure and its foundation and pertinent geologic

information. Geologic information that should be considered includes groundwater and seepage conditions; lithology, stratigraphy, and geologic details disclosed by borings, "as-built" records, and geologic interpretation; maximum past overburden at site as deduced from geologic evidence; bedding, folding and faulting; joints and joint systems; weathering; slickensides, and field evidence relating to slides, faults, movements and earthquake activity. Foundations may present problems where they contain adversely oriented joints, slickensides or fissured material, faults, seams of soft materials, or weak layers. Such defects and excess pore water pressures may contribute to instability. Special tests may be necessary to determine physical properties of particular materials. The results of stability analyses afford a means of evaluating the structure's existing resistance to failure and also the effects of any proposed modifications. Results of stability analyses should be reviewed for compatibility with performance experience when possible.

4.4.2.1. Seismic Stability. The inertial forces for use in the conventional equivalent static force method of analysis should be obtained by multiplying the weight by the seismic coefficient and should be applied as a horizontal force at the center of gravity of the section or element. The seismic coefficients suggested for use with such analyses are listed in Figures 1 through 4. Seismic stability investigations for all high hazard category dams located in Seismic Zone 4 and high hazard dams of the hydraulic fill type in Zone 3 should include suitable dynamic procedures and analyses. Dynamic analyses for other dams and higher seismic coefficients are appropriate if in the judgment of the investigating engineer they are warranted because of proximity to active faults or other reasons. Seismic stability investigations should utilize "state-of-the-art" procedures involving seismological and geological studies to establish earthquake parameters for use in dynamic stability analyses and, where appropriate, the dynamic testing of materials. Stability analyses may be based upon either time-history or response spectra techniques. The results of dynamic analyses should be assessed on the basis of whether or not the dam would have sufficient residual integrity to retain the reservoir during and after the greatest or most adverse earthquake which might occur near the project location.

4.4.2.2. Clay Shale Foundation. Clay shale is a highly overconsolidated sedimentary rock comprised predominantly of clay minerals, with little or no cementation. Foundations of clay shales require special measures in stability investigations. Clay shales, particularly those containing montmorillonite, may be highly susceptible to expansion and consequent loss of strength upon unloading. The shear strength and the resistance to deformation of clay shales may be quite low and high pore water pressures may develop under increase in load. The presence of slickensides in clay shales is usually an indication of low shear stength. Prediction

of field behavior of clay shales should not be based solely on results of conventional laboratory tests since they may be misleading. The use of peak shear strengths for clay shales in stability analyses may be unconservative because of nonuniform stress distribution and possible progressive failures. Thus the available shear resistance may be less than if the peak shear strength were mobilized simultaneously along the entire failure surface. In such cases, either greater safety factors or residual shear strength should be used.

4.4.3. Embankment Dams.

4.4.3.1. Liquefaction. The phenomenon of liquefaction of loose, saturated sands and silts may occur when such materials are subjected to shear deformation or earthquake shocks. The possibility of liquefaction must presently be evaluated on the basis of empirical knowledge supplemented by special laboratory tests and engineering judgment. The possibility of liquefaction in sands diminishes as the relative density increases above approximately 70 percent. Hydraulic fill dams in Seismic Zones 3 and 4 should receive particular attention since such dams are susceptible to liquefaction under earthquake shocks.

4.4.3.2. Shear Failure. Shear failure is one in which a portion of an embankment or of an embankment and foundation moves by sliding or rotating relative to the remainder of the mass. It is conventionally represented as occurring along a surface and is so assumed in stability analyses, although shearing may occur in a zone of substantial thickness. The circular arc or the sliding wedge method of analyzing stability, as pertinent, should be used. The circular arc method is generally applicable to essentially homogeneous embankments and to soil foundations consisting of thick deposits of fine-grained soil containing no layers significantly weaker than other strata in the foundation. The wedge method is generally applicable to rockfill dams and to earth dams on foundations containing weak layers. Other methods of analysis such as those employing complex shear surfaces may be appropriate depending on the soil and rock in the dam and foundation. Such methods should be in reputable usage in the engineering profession.

4.4.3.3. Loading Conditions. The loading conditions for which the embankment structures should be investigated are (I) Sudden drawdown from spillway crest elevation or top of gates, (II) Partial pool, (III) Steady state seepage from spillway crest elevation or top of gate elevation, and (IV) Earthquake. Cases I and II apply to upstream slopes only; Case III applies to downstream slopes; and Case IV applies to both upstream and downstream slopes. A summary of suggested strengths and safety factors are shown in Table 4.

TABLE 4

FACTORS OF SAFETY †

Case	Loading Condition	Factor of Safety	Shear †† Strength	Remarks
I	Sudden drawdown from spillway crest or top of gates to minimum drawdown elevation.	1.2*	Minimum composite of R and S shear strengths See Figure 5.	Within the drawdown zone submerged unit weights of materials are used for computing forces resisting sliding and saturated unit weights are used for computing forces contributing to sliding.
II	Partial pool with assumed horizontal steady seepage saturation.	1.5	$\frac{R+S}{2}$ for $R < S$ S for $R > S$	Composite intermediate envelope of R and S shear strengths. See Figure 6.
III	Steady seepage from spillway crest or top of gates with $K_h/K_v = 9$ assumed**	1.5	Same as Case II	
IV	Earthquake (Cases II and III with seismic loading)	1.0	***	See Figures 1 through 4 for Seismic Coefficients.

† Not applicable to embankments on clay shale foundation. Experience has indicated special problems in determination of design shear strengths for clay shale foundations and acceptable safety factors should be compatible with the confidence level in shear strength assumptions.

†† Other strength assumptions may be used if in common usage in the engineering profession.

* The safety factor should not be less than 1.5 when drawdown rate and pore water pressure developed from flow nets are used in stability analyses.

** K_h/K_v is the ratio of horizontal to vertical permeability. A minimum of 9 is suggested for use in compacted embankments and alluvial sediments.

*** Use shear strength for case analyzed without earthquake. It is not necessary to analyze sudden drawdown for earthquake loading. Shear strength tests are classified according to the controlled drainage conditions maintained during the test. R tests are those in which specimen drainage is allowed during consolidation (or swelling) under initial stress conditions, but specimen drainage is not allowed during application of shearing stresses. S tests allow full drainage during initial stress application and shearing is at a slow rate so that complete specimen drainage is permitted during the complete test.

4.4.3.4. Safety Factors. Safety factors for embankment dam stability studies should be based on the ratio of available shear strength to developed shear strength, S_D:

$$S_D = \frac{C}{F.S.} + \sigma \frac{\tan \phi}{F.S.} \tag{1}$$

C = cohesion

ϕ = angle of internal friction

σ = normal stress

The factors of safety listed in Table 4 are recommended as minimum acceptable. Final accepted factors of safety should depend upon the degree of confidence the investigating engineer has in the engineering data available to him. The consequences of a failure with respect to human life and property damage are important considerations in establishing factors of safety for specific investigations.

4.4.3.5. Seepage Failure. A critical uncontrolled underseepage or through seepage condition that develops during a rising pool can quickly reduce a structure which was stable under previous conditions, to a total structural failure. The visually confirmed seepage conditions to be avoided are (1) the exit of the phreatic surface on the downstream slope of the dam and (2) development of hydrostatic heads sufficient to create in the area downstream of the dam sand boils that erode materials by the phenomenon known as "piping" and (3) localized concentrations of seepage along conduits or through pervious zones. The dams most susceptible to seepage problems are those built of or on pervious materials of uniform fine particle size, with no provisions for an internal drainage zone and/or no underseepage controls.

4.4.3.6. Seepage Analyses. Review and modifications to original seepage design analyses should consider conditions observed in the field inspection and piezometer instrumentation. A seepage analysis should consider the permeability ratios resulting from natural deposition and from compaction placement of materials with appropriate variation between horizontal and vertical permeability. An underseepage analysis of the embankment should provide a critical gradient factor of safety for the maximum head condition of not less than 1.5 in the area downstream of the embankment.

$$F.S = i_c/i = \frac{H_c/D_b}{H/D_b} = D_b \frac{(\gamma_m - \gamma_w)}{H \gamma_w} \tag{2}$$

i_c = Critical gradient

i = Design gradient

H = Uplift head at downstream toe of dam measured above tailwater

H_c = The critical uplift

D_b = The thickness of the top impervious blanket at the downstream toe of the dam

γ_m = The estimated saturated unit weight of the material in the top impervious blanket

γ_w = The unit weight of water

Where a factor of safety less than 1.5 is obtained the provision of an underseepage control system is indicated. The factor of safety of 1.5 is a recommended minimum and may be adjusted by the responsible engineer based on the competence of the engineering data.

4.4.4. Concrete Dams and Appurtenant Structures.

4.4.4.1. Requirements for Stability. Concrete dams and structures appurtenant to embankment dams should be capable of resisting overturning, sliding and overstressing with adequate factors of safety for normal and maximum loading conditions.

4.4.4.2. Loads. Loadings to be considered in stability analyses include the water load on the upstream face of the dam; the weight of the structure; internal hydrostatic pressures (uplift) within the body of the dam, at the base of the dam and within the foundation; earth and silt loads; ice pressure, seismic and thermal loads, and other loads as applicable. Where tailwater or backwater exists on the downstream side of the structure it should be considered, and assumed uplift pressures should be compatible with drainage provisions and uplift measurements if available. Where applicable, ice pressure should be applied to the contact surface of the structure at normal pool elevation. A unit pressure of not more than 5,000 pounds per square foot should be used. Normally, ice thickness should not be assumed greater than two feet. Earthquake forces should consist of the inertial forces due to the horizontal acceleration of the dam itself and hydrodynamic forces resulting from the reaction of the reservoir water against the structure. Dynamic water pressures for use in conventional methods of analysis may be computed by means of the "Westergaard Formula" using the parabolic approximation (H.M. Westergaard, "Water Pressures on Dams During Earthquakes," Trans., ASCE, Vol 98, 1933, pages 418-433), or similar method.

4.4.4.3. Stresses. The analysis of concrete stresses should be based on in situ properties of the concrete and foundation. Computed maximum compressive stresses for normal operating conditions in the order of 1/3 or less of in situ strengths should be satisfactory. Tensile stresses in unreinforced concrete should be acceptable only in locations where cracks will not adversely affect the overall performance and stability of the structure. Foundation stresses should be such as to provide adequate safety against failure of the foundation material under all loading conditions.

4.4.4.4. Overturning. A gravity structure should be capable of resisting all overturning forces. It can be considered safe against overturning if the resultant of all combinations of horizontal and vertical forces, excluding earthquake forces, acting above any horizontal plane through the structure or at its base is located within the middle third of the section. When earthquake is included the resultant should fall within the limits of the plane or base, and foundation pressures must be acceptable. When these requirements for location of the resultant are not satisfied the investigating engineer should assess the importance to stability of the deviations.

4.4.4.5. Sliding. Sliding of concrete gravity structures and of abutment and foundation rock masses for all types of concrete dams should be evaluated. by the shear-friction resistance concept. The available sliding resistance is compared with the driving force which tends to induce sliding to arrive at a sliding stability safety factor. The investigation should be made along all potential sliding paths. The critical path is that plane or combination of planes which offers the least resistance.

4.4.4.5.1. Sliding Resistance. Sliding resistance is a function of the unit shearing strength at no normal load (cohesion) and the angle of friction on a potential failure surface. It is determined by computing the maximum horizontal driving force which could be resisted along the sliding path under investigation. The following general formula is obtained from the principles of statics and may be derived by resolving forces parallel and perpendicular to the sliding plane:

$$R_R = V \tan (\phi + \alpha) + \frac{cA}{\cos\alpha (1 - \tan \phi \tan\alpha)} \qquad (3)$$

where

R_R = Sliding Resistance (maximum horizontal driving force which can be resisted by the critical path)

ϕ = Angle of internal friction of foundation material or, where applicable, angle of sliding friction

V = Summation of vertical forces (including uplift)

c = Unit shearing strength at zero normal loading along potential failure plane

A = Area of potential failure plane developing unit shear strength "c"

α = Angle between inclined plane and horizontal (positive for uphill sliding)

For sliding downhill the angle α is negative and Equation (1) becomes:

$$R_R = V \tan (\phi - \alpha) + \frac{cA}{\cos\alpha (1 + \tan \phi \tan\alpha)} \qquad (4)$$

When the plane of investigation is horizontal, and the angle α is zero and Equation (1) reduced to the following:

$$R_R = V \tan \phi + cA \qquad (5)$$

4.4.4.5.2. Downstream Resistance. When the base of a concrete structure is embedded in rock or the potential failure plane lies below the base, the passive resistance of the downstream layer of rock may sometimes be utilized for sliding resistance. Rock that may be subjected to high velocity water scouring should not be used. The magnitude of the downstream resistance is the lesser of (a) the shearing resistance along the continuation of the potential sliding plane until it daylights or (b) the resistance available from the downstream rock wedge along an inclined plane. The theoretical resistance offered by the passive wedge can be computed by a formula equivalent to formula (3):

$$P_p = W \tan(\phi + \alpha) + \frac{cA}{\cos\alpha\,(1 - \tan\phi\,\tan\alpha)} \qquad (6)$$

P_p = passive resistance of rock wedge

W = weight (buoyant weight if applicable) of downstream rock wedge above inclined plane of resistance, plus any superimposed loads

ϕ = angle of internal friction or, if applicable, angle of sliding friction

α = angle between inclined failure plane and horizontal

c = unit shearing strength at zero normal load along failure plane

A = area of inclined plane of resistance

When considering cross-bed shear through a relatively shallow, competent rock strut, without adverse jointing or faulting, W and α may be taken at zero and 45°, respectively, and an estimate of passive wedge resistance per unit width obtained by the following equation:

$$P_p = 2\,cD \qquad (7)$$

where

D = Thickness of the rock strut

4.4.4.5.3. Safety Factor. The shear-friction safety factor is obtained by dividing the resistance R_R by H, the summation of horizontal service

loads to be applied to the structure:

$$S_{s-f} = \frac{R_R}{H} \tag{8}$$

When the downstream passive wedge contributes to the sliding resistance, the shear fruction safety factor formula becomes:

$$S_{s-f} = \frac{R_R + P_p}{H} \tag{9}$$

The above direct superimposition of passive wedge resistance is valid only if shearing rigidities of the foundation components are similar. Also, the compressive strength and buckling resistance of the downstream rock layer must be sufficient to develop the wedge resistance. For example, a foundation with closely spaced, near horizontal, relatively weak seams might not contain sufficient buckling strength to develop the magnitude of wedge resistance computed from the cross-bed shear strength. In this case wedge resistance should not be assumed without resorting to special treatment (such as installing foundation anchors). Computed sliding safety factors approximating 3 or more for all loading conditions without earthquake, and 1.5 including earthquake, should indicate satisfactory stability, depending upon the reliability of the strength parameters used in the analyses. In some cases when the results of comprehensive foundation studies are available, smaller safety factors may be acceptable. The selection of shear strength parameters should be fully substantiated. The bases for any assumptions; the results of applicable testing, studies and investigations; and all pre-existing, pertinent data should be reported and evaluated.

APPENDIX E

HYDRAULIC/HYDROLOGIC CALCULATIONS

MICHAEL BAKER, JR., INC.
THE BAKER ENGINEERS

Box 280
Beaver, Pa. 15009

Subject APPENDIX INDEX
S.O. No. ______
Sheet No. ______ of ______
Drawing No. ______
Computed by ______ Checked by ______ Date ______

PINES LAKE

APPENDIX

MICHAEL BAKER, JR., INC.
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Box 280
Beaver, Pa. 15009

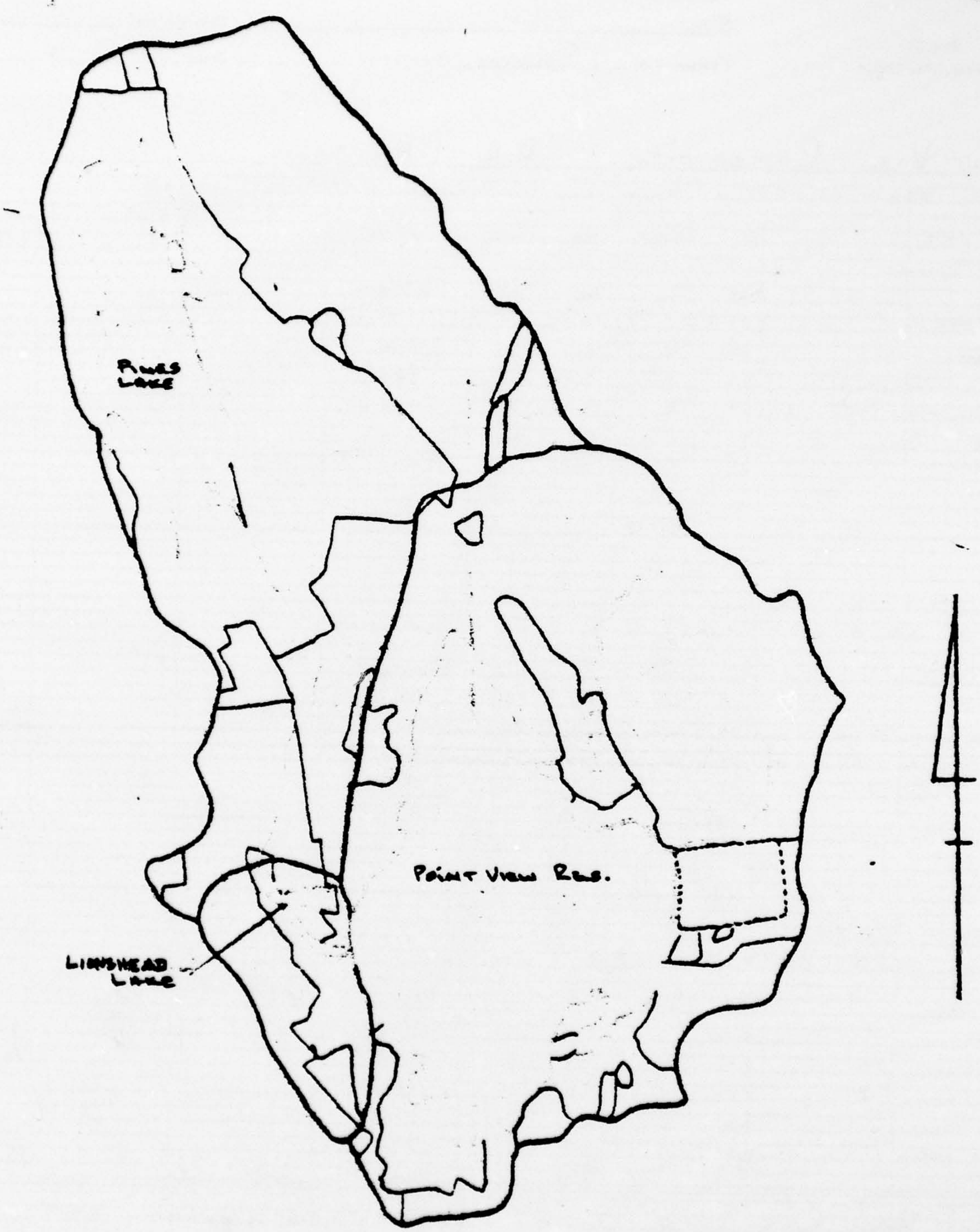
Subject N J DAM INSP.
PINES LAKE
PRECIP DATA
S.O. No.
Sheet No. 1 of 18
Drawing No.
Computed by JRM Checked by DJG Date 7-27-78

① PMP = 25.0 in - 6hr FROM Design of Small Dams (DSD)

② Drainage AREA TOTAL = 3.70 mi² ⇒ 20% REDUCTION IN PMF
UNCONTROLLED DRAINAGE AREA = 1.66 mi²

PMF = 0.8 × 25 = 20 INCHES

③ PRECIP PATTERN By CORPS OF ENGS EC-1110-2-163
FOR MOST INTENSE 6 HRS 10%, 12%, 15%, 38%, 14%, 11%
with Appropriate adjustments for Duration
The Rainfall breakdown is on pages 5-6 of this Appendix



	D.A.
POINTVIEW RESERVOIR	1.82 Sq.mi
LIONHEAD LAKE	0.22 Sq.mi
PINES LAKE	1.66 Sq.mi
WAYNE TWP., PASSAIC CO.	Total 3.70 sq.mi

NEW JERSEY

1" = 2000'

MICHAEL BAKER, JR., INC.	Subject CN VALUES FOR	S.O. No.
THE BAKER ENGINEERS	Point View Res., Lionhead Lake, Pines Lake,	Sheet No. 2 of 18
Box 280	Split Rock Res.	Drawing No.
Beaver, Pa. 15009	Computed by J. Sawyer Checked by	Date 7-23-78

POINT VIEW RESERVOIR   D.A. 1.82 sq.mi.

TOTAL AREA 12.68 $\square$" = 1.82 sq.mi.   Reservoir 4.17 $\square$"

$12.68 - 4.17 = 8.51$ $\square$"

LAND USE	$\square$"	SOIL CLASS	%	CURVE #	PRODUCT
WATER SURFACE	.05	B	0.6	100	60
WOODS	5.13	C	60.2	73	4394.6
OPEN SPACES	1.31	C	15.4	74	1139.6
CULTIVATED LAND	.53	B	6.2	71	440.2
RESIDENTIAL ½ A.	.20	B	2.4	70	168
RESIDENTIAL 1 A.	1.07	C	12.6	79	995.4
ROADS	.22	C	2.6	90	234
	8.51		100%		7431.8

CN = 74

LIONHEAD LAKE

TOTAL AREA 1.53 $\square$" = 0.22 sq.mi.   Reservoir 0.13 $\square$"

$1.53 - .13 = 1.40$ $\square$"

LAND USE	$\square$"	SOIL CLASS	%	CURVE #	PRODUCT
WOODS	.43	C	30.7	73	2241.1
RESIDENTIAL ½ A.	.79	C	56.4	80	4512.
RESIDENTIAL 1 A.	.10	C	7.2	79	568.8
ROADS (HARD)	.08	C	5.7	90	513.
	1.40		100%		7834.9

CN = 78

PINES LAKE

TOTAL AREA 11.57 $\square$" = 1.66 sq.mi.   Reservoir 1.46 $\square$"

$11.57 - 1.46 = 10.11$ $\square$"

LAND USE	$\square$"	SOIL CLASS	%	CURVE #	PRODUCT
WOODS	5.30	C	52.4	73	3825.2
RESIDENTIAL ½ A.	3.79	C	37.5	80	3000.
RESIDENTIAL 1 A.	0.22	C	2.2	79	173.8
OPEN SPACES	0.41	C	4.0	74	296.
ROADS (HARD)	0.39	C	3.9	90	351.
	10.11		100%		7646.0

CN = 76

SPLIT ROCK RESERVOIR

TOTAL AREA 39.96 $\square$" = 5.59 sq.mi., Reservoir 5.70 $\square$"

$39.96 - 5.70 = 34.26$ $\square$"

LAND USE	$\square$"	SOIL CLASS	%	CURVE #	PRODUCT
WOODS	30.85	B	89.7	60	5382.
MEADOW	1.24	B	3.6	58	208.8
ROADS [illegible]	0.16	B	0.5	82	41.
[illegible]	0.07	B	0.2	84	16.8
WATER SURFACE	1.65	B	5.0	100	500.
RESIDENTIAL ½ A.	0.32	B	1.0	70	70.
	34.26		100%		6218.6

CN = 62

MICHAEL BAKER, JR., INC.
THE BAKER ENGINEERS

Box 280
Beaver, Pa. 15009

Subject: PINES LAKE
Determination of PMF
S.O. No.
Sheet No. 4 of 18
Drawing No.
Computed by JRM Checked by REL Date 7-27-78

10, 12, 15, 38, 14, 11

PMP = 20

Min. = .12"/hr = .02"/8 CLASS = C

TIME	% Rain	Cum. %	Incr. Rain	(P) Cum. Rain	(Q) Cum. Excess	Δ Excess	Incr. Loss	Exc. Adj
0	0	0	0	0	0	0	0	0
8	1.33	1.33	0.27	0.27	0.00	0	.27	0
16	1.33	2.66	0.27	0.54	0.00	0	.27	0
24	1.33	3.99	0.27	0.81	0.01	.01	.26	.01
32	1.33	5.32	0.27	1.08	0.06	.05	.22	.05
40	1.33	6.65	0.27	1.35	0.13	.07	.20	.07
48	1.33	7.98	0.27	1.62	0.24	.11	.16	.11
56	1.33	9.31	0.27	1.89	0.36	.12	.15	.12
64	1.47	10.78	0.29	2.18	0.51	.15	.14	.15
72	1.60	12.38	0.32	2.50	0.70	.19	.13	.19
80	1.60	13.98	0.32	2.82	0.90	.20	.12	.20
88	1.60	15.58	0.32	3.14	1.11	.21	.11	.21
96	1.60	17.18	0.32	3.46	1.34	.23	.09	.23
104	1.60	18.78	0.32	3.78	1.57	.23	.09	.23
112	1.60	20.38	0.32	4.10	1.82	.25	.07	.25
4.9 hrs 120	1.60	22.0	0.32	4.42	2.07	.25	.07	.25
128	2.0	24.0	0.40	4.82	2.39	.32	.08	.32
136	2.0	26.0	0.40	5.22	2.72	.33	.07	.33
144	2.0	28.0	0.40	5.62	3.06	.34	.06	.34
152	2.0	30.0	0.40	6.02	3.40	.34	.06	.34
160	2.0	32.0	0.40	6.42	3.75	.35	.05	.35
168	2.0	34.0	0.40	6.82	4.10	.35	.05	.35
176	2.0	36.0	0.40	7.22	4.45	.35	.05	.35
184	3.53	39.53	0.71	7.93	5.09	.64	.07	.64
192	5.07	44.60	1.01	8.94	6.02	.93	.08	.93
200	5.07	49.67	1.01	9.95	6.96	.94	.07	.94

MICHAEL BAKER, JR., INC.
THE BAKER ENGINEERS

Box 280
Beaver, Pa. 15009

Subject: PINES LAKE
Determination of PMF

S.O. No.
Sheet No. 5 of 18
Drawing No.

Computed by: JRM   Checked by: CEd   Date: 7-27-78

	TIME	%	Cum %	Incr. Rain	(P) Cum. Rain	(Q) Cum. Exc.	Δ Excess	Incr. Loss	Exc. Adj.
	208	5.07	54.74	1.01	10.96	7.91	0.95	.06	.95
	216	5.07	59.81	1.01	11.97	8.87	.96	.05	.96
	224	5.07	64.88	1.01	12.98	9.83	.96	.05	.96
	232	5.07	69.94	1.01	13.99	10.80	.97	.04	.97
4hrs	240	5.07	75.00	1.01	15.00	11.78	.98	.03	.98
	248	1.87	76.87	0.37	15.37	12.14	.36	.01 .02	.35
	256	1.87	78.74	0.38	15.75	12.51	.37	.01 .02	.36
	264	1.87	80.61	0.37	16.12	12.87	.36	.01 .02	.35
	272	1.87	82.48	0.37	16.49	13.23	.36	.01 .02	.35
	280	1.87	84.37	0.38	16.87	13.59	.36	.02 .02	.36
	288	1.87	86.24	0.37	17.24	13.96	.37	.00 .02	.35
	296	1.87	88.11	0.38	17.62	14.33	.37	.01 .02	.36
	304	1.66	89.77	0.33	17.95	14.65	.32	.01 .02	.31
	312	1.47	91.24	0.29	18.24	14.93	.28	.01 .02	.27
	320	1.47	92.71	0.30	18.54	15.22	.29	.01 .02	.28
	328	1.47	94.17	0.29	18.83	15.51	.29	.00 .02	.27
	336	1.47	95.64	0.30	19.13	15.80	.29	.01 .02	.28
	344	1.47	97.11	0.29	19.42	16.08	.28	.01 .02	.27
	352	1.47	98.58	0.29	19.71	16.37	.29	.00 .02	.27
6hrs	360	1.47	100.00	0.29	20.00	16.65	.28	.01 .02	.27
									16.48

MICHAEL BAKER, JR., INC.
THE BAKER ENGINEERS

Box 280
Beaver, Pa. 15009

Subject: PINES LAKE DAM
Determination of 1/2 PMF

S.O. No.
Sheet No. 6 of 18
Drawing No.
Computed by JRM Checked by REH Date 8-7-78

TIME	% Rain	Cum %	Incr. Rain (Try 11")	Cum. Rain (P)	Cum. Excess (Q)	Excess	Incr. Loss	Exc Adj
0			0	0	0	0		
8			0.15	0.15		0		
16			0.15	0.30		0		
24			0.15	0.45		0		
32			0.15	0.60		0		
40			0.15	0.75	0.00	0		
48			0.15	0.90	0.02	.02		
56			0.15	1.05	0.05	.03		
64			0.16	1.21	0.09	.04		
72			0.18	1.39	0.15	.06		
80			0.18	1.57	0.22	.07		
88			0.18	1.75	0.29	.07		
96			0.18	1.93	0.38	.09		
104			0.18	2.11	0.47	.11 0.09		
112			0.18	2.29	0.57	.10		
120			0.18	2.47	0.68	.11		
128			0.22	2.69	0.81	.13		
136			0.22	2.91	0.96	.15		
144			0.22	3.13	1.10	.14		
152			0.22	3.35	1.26	.16		
160			0.22	3.57	1.42	.16		
168			0.22	3.79	1.58	.16		
176			0.22	4.01	1.75	.17		
184			0.38	4.39	2.05	.30		
192			0.55	4.94	2.50	.45		
200			0.55	5.49	2.97	.47		

MICHAEL BAKER, JR., INC.
THE BAKER ENGINEERS

Box 280
Beaver, Pa. 15009

Subject: PINES LAKE DAM
Determination of [illegible]

S.O. No.
Sheet No. 7 of 18
Drawing No.

Computed by: JRM Checked by: REK Date: 8-7-79

.02 min

TIME	% Rain	Cum %	Incr. Rain	(P) Cum Rain	(Q) Cum Excess	Δ Excess	Incr Loss		Exc Adj
208			0.55	6.04	3.42	.45	.10		.45
216			0.55	6.59	3.89	.47	.08		.47
224			0.55	7.14	4.38	.49	.06		.47
232			0.55	7.69	4.88	.50	.05		.50
240			0.55	8.24	5.38	.50	.05		.50
248			0.21	8.43	5.55	.17	.04		.17
256			0.21	8.64	5.74	.19	.02		.17
264			0.21	8.85	5.94	.20	.01	.02	.19
272			0.21	9.06	6.13	.19	.02		.19
280			0.21	9.27	6.33	.20	.01	.02	.19
288			0.21	9.48	6.52	.19	.02		.19
296			0.21	9.69	6.72	.20	.01	.02	.19
304			0.18	9.87	6.89	.17	.01	.02	.16
312			0.16	10.03	7.04	.15	.01	.02	.14
320				10.19	7.19	.15	.01	.02	.14
328				10.35	7.34	.15	.01	.02	.14
336				10.51	7.49	.15	.01	.02	.14
344				10.67	7.64	.15	.01	.02	.14
352				10.83	7.79	.15	.01	.02	.14
360			0.17	11.00	7.95	.16	.01		.15
				11.11					7.86
									7.89

MICHAEL BAKER, JR., INC.
THE BAKER ENGINEERS

Box 280
Beaver, Pa. 15009

Subject: Pines Lake — S.O. No. ___
Time of Concentration TR55 — Sheet No. 8 of 18
From Urban Hydrology for Small Watersheds — Drawing No. ___
Computed by ALB — Checked by ___ — Date ___

SLOPE OF STREAM:		VELOCITY OF STREAM	
260-300	2.963 %	260-300	3.45 ft/sec
300-360	5.825 %	300-360	4.75 ft/sec
360-400	2.899 %	360-400	3.45 ft/sec
400-460	2.667 %	400-460	2.90 ft/sec
460-620	9.581 %	460-620	0.78 ft/sec

TRAVEL TIME OF WATER

260-300	391.30 SEC	= 0.11 HR ✓
300-360	~~212.84~~ 227 SEC	= 0.06 HR ✓
360-400	400.00 SEC.	= 0.11 HR
400-460	775.86 SEC	= 0.22 HR
460-620	2141.00 SEC.	= 0.59 HR
		7.09 HR ✓ = Tc

MICHAEL BAKER, JR., INC.
THE BAKER ENGINEERS

Box 280
Beaver, Pa. 15009

Subject PINES LAKE S.O. No.

Development of T_c, D, t_p Sheet No. 9 of 18

Drawing No.

Computed by JRM Checked by Date 7-27-78

Total Area = 1.66 sq. mi., CLASS "C" soil, CN = 76

Design of Small Dams, page 70

ELEV.	Slope of Stream %	Velocity ft/s	Length (ft)	Time of Concentration (sec)
620-460	9.6	3 ft/s Tex	1670	556.67 ✓
460-400	2.7	1.5 Tex	2250	1500.00 ✓
400-360	2.9	3.0	1380	460.00 ✓
360-300	5.8	4.0	1080	~~257.50~~ 270
300-260	2.9	3.0	1350	450.00 ✓
			7680	~~3224.17~~ 3237 sec = 0.90 hr = T_c

Using the T_c from Technical Release #55 = 1.1 hr.

An average of the two values is 1.0 hr.

Use T_c = 1.0 hours.

$D = 8$ minutes = (.13 hr)

$t_p = \frac{.13}{2} + 0.6(1) = .67$ hr. = 40 minutes

USE T_c = 1.00 hr, D = .13 hr = 8 min., t_p = .67 hr = 40 min.

$q_p = \frac{484\,(A)\,Q}{T_p} = \frac{484\,(1.66)(1)}{.67} = 1199 = 1200$ cfs ✓

MICHAEL BAKER, JR., INC.
THE BAKER ENGINEERS

Box 280
Beaver, Pa. 15009

Subject: PINES LAKE
S.O. No.
Sheet No. 10 of 18
Inflow Hydrograph
Drawing No.
Computed by JRM
Checked by REH
Date 7-27-78

$T_p = .67 hr$ $q_p = 1200$ cfs

From Design of Small Dams
Curvalinear Adj to SCS
Triangular U.H.

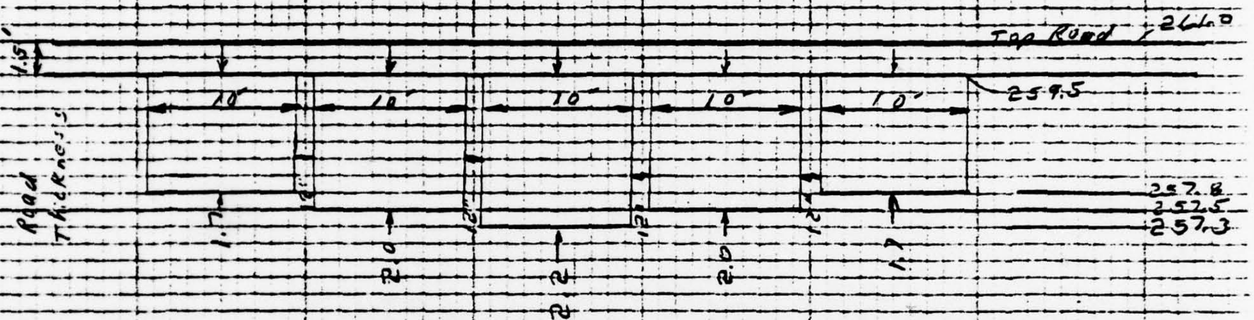
T/T_p	T (hr)	q/q_p	q	q Rev
0	0	0	0	0
0.2	.13	.075	90	92
0.4	.27	.28	336	344
0.6	.40	.60	720	736
0.8	.54	.89	1068	1092
1.0	.67	1.00	1200	1227
1.2	.80	.92	1104	1129
1.4	.94	.75	900	920
1.6	1.07	.56	672	687
1.8	1.21	.42	504	515
2.0	1.34	.32	384	393
2.2	1.47	.24	288	295
2.4	1.61	.18	216	221
2.6	1.74	.13	156	160
2.8	1.88	0.098	118	121
3.0	2.01	0.075	90	92
3.2		.056	67	69
3.4		.0425	51	52
3.6		.0325	39	40
3.8		.025	30	31
4.0	2.68	.019	22	23
4.2		.014	17	17
4.4		.011	13	13
4.6		.008	10	10
4.8		.005	6	6
5.0	3.35	.004	5	5
			8106	

$$\text{Revision} = 8106 \frac{(.13)(3600)(12)}{5280^2 (1.67)} = 0.98$$

MICHAEL BAKER, JR., INC.
THE BAKER ENGINEERS

Box 280
Beaver, Pa. 15009

Subject PINES LAKE DAM
SPILLWAY RATING
S.O. No.
Sheet No. 10 of 18
Drawing No.
Computed by JRM Checked by REL Date 8-4-78



Breadth of Spillway is 3.3' - taken from reproduced plan

Broad-crested weir

Used an average opening of 2.0'

$L = L' - 2(NK_p + K_a)H_e$

$L = 50 - 2[4(.02) + .2]H_e$

$L = 50 - .56 H_e$

$Q = CLH_e^{3/2}$

Design of Small Dam - pg 373

K_p (square nosed piers)

K_a (square abutment)

Elev.	Head	Weir $H_e^{3/2}$	L	C	Q cfs
257.3	0	0	0		0
257.5	0 (.2)	(0.089)	(~~9.89~~ 10)	(2.42)	(2)
257.7	.2	0.089	49.9	2.42	11
258.0	.5	0.35	49.7	2.65	47
258.5	1.0	1.0	49.4	2.66	131
259.0	1.5	1.84	49.2	2.66	240
259.5	2.0	2.83	48.9	2.71	375

MICHAEL BAKER, JR., INC.
THE BAKER ENGINEERS

Box 280
Beaver, Pa. 15009

Subject: PINES LAKE DAM
RATING CURVE EXTENSION
ABOVE TOP OF DAM

S.O. No.

Drawing No.

Computed by DJG Checked by REH Date 8/4/78

RATING ABOVE TOP OF DAM

L = 228' = Breadth = 8' Using Broad Crested Weir Formula

*Use 10' Breadth values from Handbook of Hydraulics $Q = CLH^{3/2}$

	Elev.	H (FT)	C*	Q CFS
Top Roadway	261.0	0	0	0
	~~261.5~~	~~0.5~~	~~2.7~~	~~218~~
	261.6	0.6	2.7	286 ✓
	262.0	1.0	2.68	611 ✓
	262.6	1.6	2.64	1218 ✓
	263.0	2.0	2.64	1702 ✓

MICHAEL BAKER, JR., INC.
THE BAKER ENGINEERS

Box 280
Beaver, Pa. 15009

Subject: Pines Lake Dam
Spillway Rating - Orifice
Flow Caused by Bridge

S.O. No.

Drawing No.

Computed by: REH & JRM

Checked by:

Date: 8/24/78

Orifice Flow Equation:

$Q = CA_r\sqrt{2gh}$ where: h = distance from water surface to center of orifice

C = 0.61 for sharp-edged orifice

Average opening = 2.0 feet

Elev. (feet)	Depth behind Road (ft)	Area (ft²)	h (feet)	Q (cfs)
259.7	0.2	100	1.2	536
260.0	0.5	100	1.5	600
261.0	1.5	100	2.5	774
261.5	2.0	100	3.0	348
262.0	2.5	100	3.5	915
262.6	3.1	100	4.1	991
263.0	3.5	100	4.5	1038

MICHAEL BAKER, JR., INC.

THE BAKER ENGINEERS

Box 280

Beaver, Pa. 15009

Subject: PINES LAKE

Spillway rating

S.O. No.

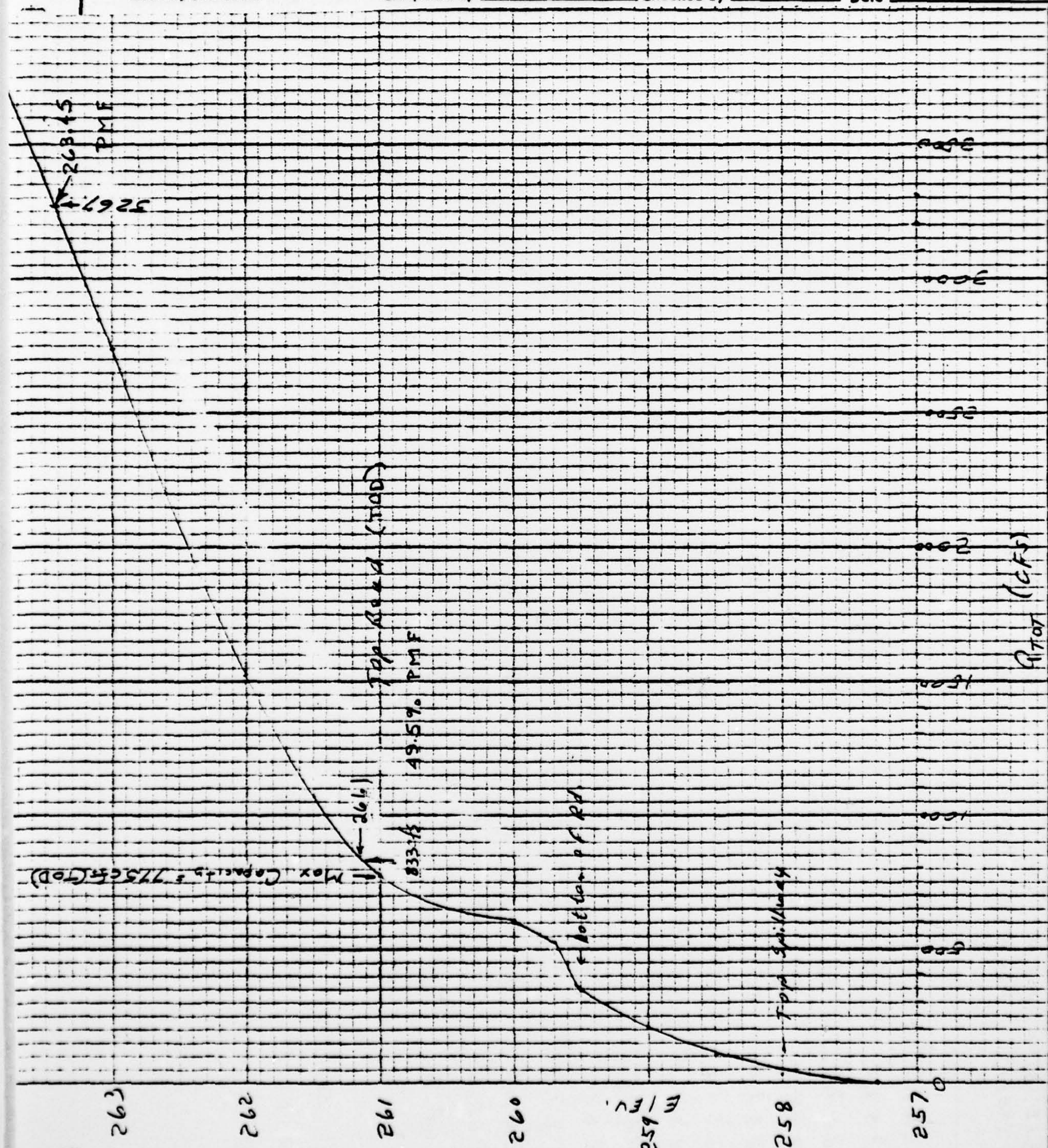
Sheet No. 13 of 18

Drawing No.

Computed by JRM

Checked by REH

Date 8-4-78



MICHAEL BAKER, JR., INC.
THE BAKER ENGINEERS
Box 280
Beaver, Pa. 15009

Subject: PINES LAKE
STAGE - DISCHARGE - STORAGE
S.O. No.

Drawing No.
Computed by JRM   Checked by REK   Date

ELEVATION (FEET)	WEIR FLOW (CFS)	ORIFICE FLOW (CFS)	Total DISCHARGE (CFS)	STORAGE (AC-FT)
257.3 (1)	0	0	0	0
257.5	2	0	2	25
257.7	11	0	11	50
258.0	47	0	47	91
258.5	131	0	131	163
259.0	240	0	240	235
259.5 (2)	375	0	375	305
259.7	—	536	536	333
260.0	—	600	600	375
261.0 (3)	0	774	774	520
261.5	218	848	1066	590
262.0	611	915	1526	660
262.6	1218	991	2209	740
263.0	1702	1038	2740	800

(1) Spillway Crest
(2) Bottom of Road, orifice flow controls
(3) Top of Dam, weir flow over top of dam is calculated

MICHAEL BAKER, JR., INC.
THE BAKER ENGINEERS

Box 280
Beaver, Pa. 15009

Subject: Pine Lake — S.O. No.

Elevation vs. Surface Area —

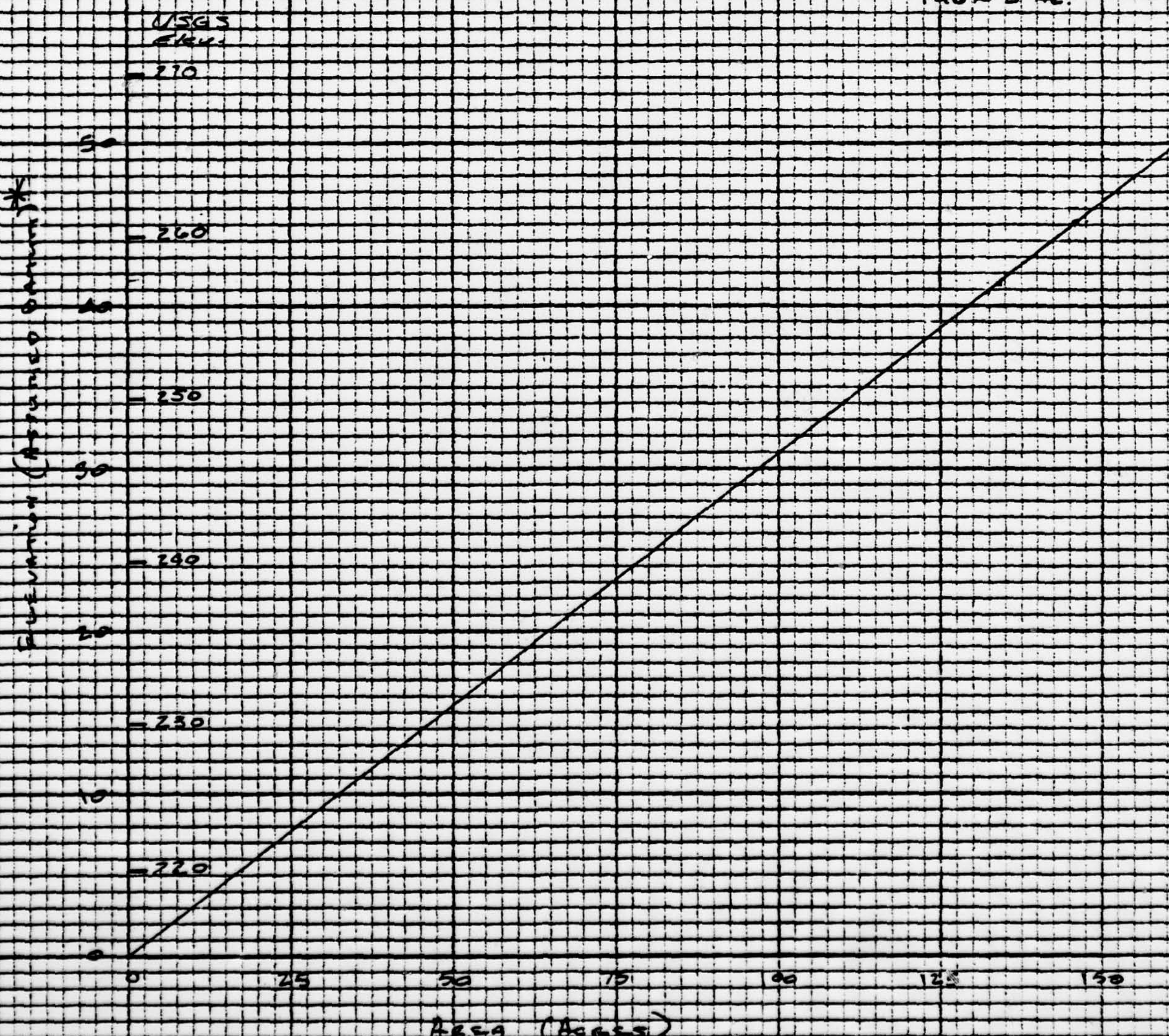
Drawing No.

Computed by J Sawyer — Checked by — Date

NOTE: Bottom of Lake elevation unknown
Datum assumed 0.00
Dam height known therefore computations
will be the same but based on assumed datum

Dam Height = 41.5'

Area of Pine Lake at Normal Pool = 134.07 Ac. — 45.2' top of Dam 146.2 Ac.

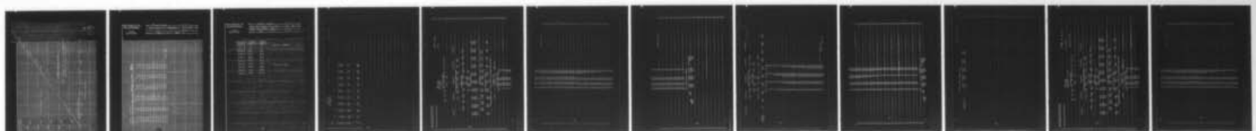


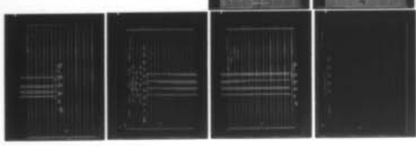
* USGS ELEVATION = Assumed Elev. + 215.8

AD-A059 831 BAKER (MICHAEL) JR INC BEAVER PA F/G 13/2
NATIONAL DAM SAFETY PROGRAM. PINFS LAKE DAM (NJ00250), PASSAIC --ETC(U)
AUG 78 M BAKER DACW61-78-C-0141

UNCLASSIFIED NL

2 OF 2
AD
A059831





END
DATE
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12-78
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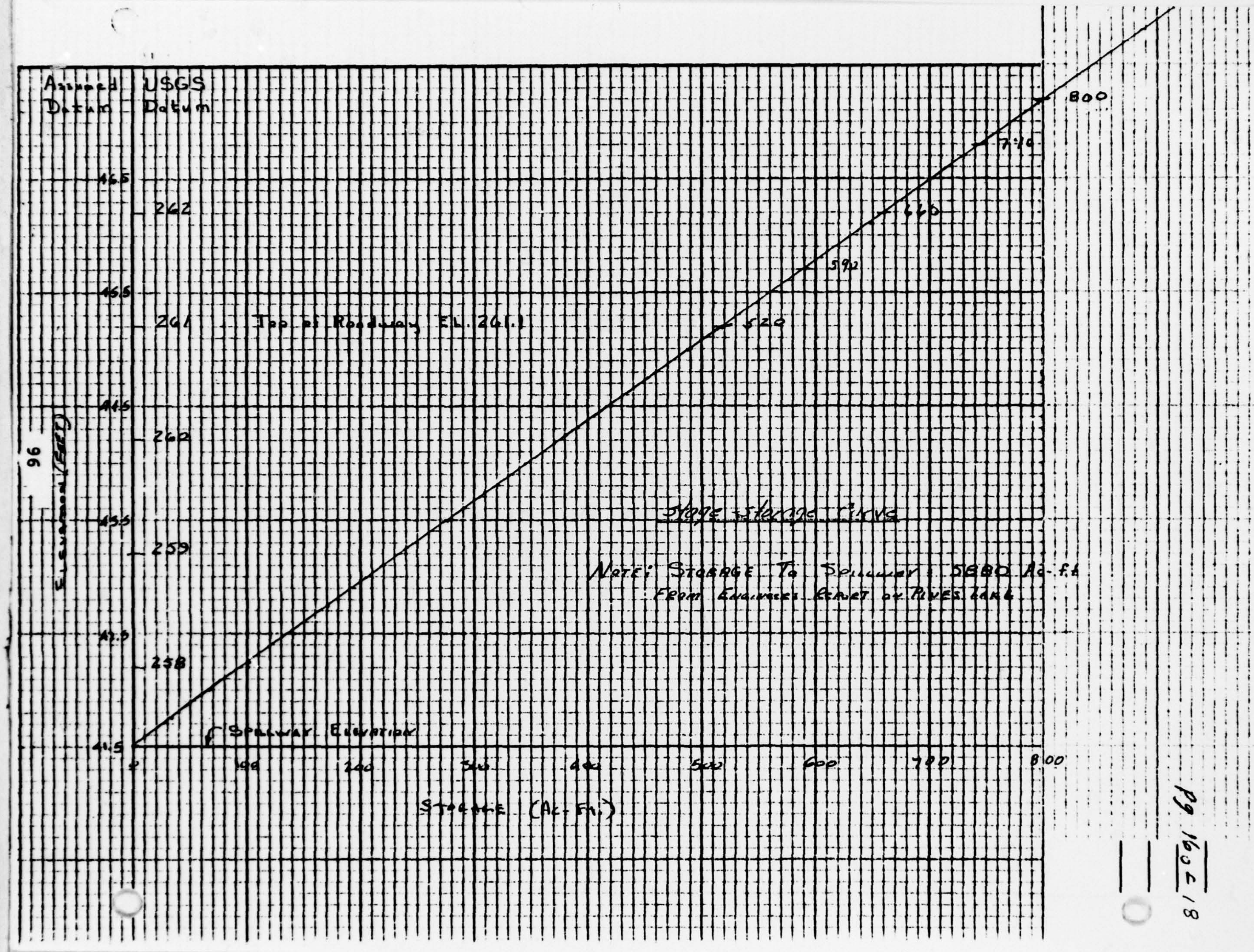
Assumed Datum
USGS Datum
Elevation (Feet)
46.5
45.5
44.5
43.5
42.5
41.5
262
261
260
259
258
Top of Roadway EL. 261.1
Spillway Elevation
800
740
660
592
520
Stage Storage Curve
Note: Storage To Spillway: 5880 Ac-ft.
From Engineers Report by Pines Lake
0
100
200
300
400
500
600
700
800
Storage (Ac-Ft.)

Pg 16 of 18

MICHAEL BAKER, JR., INC.
THE BAKER ENGINEERS

Box 280
Beaver, Pa. 15009

Subject: Pines Lake — Elevation vs Storage

S.O. No. ____

Sheet No. 17 of 18

Drawing No. ____

Computed by: J. Sawyer   Checked by: REH   Date: ____

USGS Elevation	Assumed Elevation	Surface Area	Average Surface Area	Inc. Storage (Ac-Ft)	Cum. Storage (Ac-Ft)	
257.3	41.5	134.27				
257.55	41.75	134.38	134.48	33.62	33.62	
257.80	42.0	135.69	135.29	33.82	67.44	
258.05	42.25	136.49	136.09	34.02	101.46	
258.3	42.5	137.32	136.90	34.23	135.69	
258.55	42.75	138.1	137.71	34.43	170.12	
258.80	43.0	138.92	138.53	34.63	204.75	
259.05	43.25	139.72	139.33	34.83	239.58	
259.30	43.5	140.53	140.13	35.03	274.61	
259.53	43.75	141.38	140.94	35.24	309.85	
259.80	44.0	142.15	141.75	35.44	345.29	
260.05	44.25	142.96	142.56	35.64	380.93	
260.30	44.5	143.76	143.36	35.84	416.77	
260.55	44.75	144.57	144.17	36.04	452.81	
260.80	45.0	145.38	144.98	36.25	489.06	
261.00	45.2	146.22	145.70	29.14	518.20	TOP OF DAM

MICHAEL BAKER, JR., INC.
THE BAKER ENGINEERS

Box 280
Beaver, Pa. 15009

Subject PINES LAKE
elevation-storage-discharge
USED FOR HEC-1 RUN
Computed by JEM Checked by
S.O. No.
Sheet No. 18 of 18
Drawing No.
Date

ELEVATION (FT)	STORAGE (AC-FT)	OUTFLOW (CFS)	
257.3	0	0	SPILLWAY CREST
257.8	67	20	
258.8	205	160	
259.5	305	375	
259.7	333	536	
260.0	375	600	
261.0	520	774	TOD EL 261.1
261.5	590	1066	
262.0	660	1526	
263.0	800	2740	

```
A                           PINES LAKE               HEC-1   INPUT LISTING                Sheet 1 of 3
A                           NEW JERSEY
A                           DAM ANALYSIS
B    100              8
1      3
K             1                         2         1
M            -1     1.7
O     45
1      0      C     .01     .05       .07       .11       .12       .15       .19       .20
1    .21    .23     .23     .25       .25       .32       .33       .34       .34       .35
1    .35    .35     .64     .93       .94       .95       .96       .96       .97       .98
1    .35    .36     .35     .35       .36       .35       .36       .31       .27       .28
1    .27    .28     .27     .27       .27
T
U     25
1     92    344     736    1092      1227      1129       920       689       513       343
1    255    221     160     121        92        65        52        40        31        23
1     17     13      10       6         5
X                     1
K      1      1                         2         1
Y                             1
1      1
2      0     67     205     305       330       375       520       590       660       800
3      0     20     160     345       536       600       774      1066      1526      2740
K     99
A
```

Sheet 2 of 13

PMF

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******************************
HEC-1 VERSION DATED JAN 1973

******************************

                                   PINES LAKE
                                   NEW JERSEY
                                   DAM ANALYSIS

                                 JOB SPECIFICATION
                  NQ    NHR   NMIN   IDAY   IHR   IMIN METRC   IPLT   IPRT NSTAN
                 100      0      8      0     0      0     0      0      0     0
                                    JOPER     NWT
                                        3       0


        **********        **********        **********        **********        **********

                            SUB-AREA RUNOFF COMPUTATION
                  ISTAQ    ICOMP    IECON    ITAPE     JPLT     JPRT    INAME
                      1        0        0        0        2        1        0

                                  HYDROGRAPH DATA
          IHYDG     IUHG    TAREA     SNAP    TRSDA    TRSPC    RATIO    ISNOW    ISAME    LOCAL
              0       -1     1.70      0.0      0.0      0.0      0.0        0        0        0

                                    PRECIP DATA
                               NP    STORM      DAJ      DAK
                               45      0.0      0.0      0.0
                                  PRECIP PATTERN
     0.0      0.0      0.01     0.05     0.07     0.11     0.12     0.15     0.19     0.20
     0.21     0.23     0.23     0.25     0.25     0.32     0.33     0.34     0.34     0.35
     0.35     0.35     0.64     0.93     0.94     0.95     0.96     0.96     0.97     0.98
     0.35     0.36     0.35     0.35     0.36     0.35     0.36     0.31     0.27     0.28
     0.27     0.28     0.27     0.27     0.27

                                     LOSS DATA
        STRKR    DLTKR    RTIOL    ERAIN    STRKS    RTIOK    STRTL    CNSTL    ALSMX    RTIMP
          0.0      0.0     1.00      0.0      0.0     1.00      0.0      0.0      0.0      0.0

                            GIVEN UNIT GRAPH, NUHGQ=  25
      92.      344.     736.    1092.    1227.    1129.     920.     689.     515.     393.
     295.      221.     160.     121.      92.      69.      52.      40.      31.      23.
      17.       13.      10.       6.       5.
                 UNIT GRAPH TOTALS   8292. CFS OR 1.01 INCHES OVER THE AREA

                                  RECESSION DATA
                     STRTQ=     0.0      QRCSN=     0.0      RTIOR= 1.00

                                END-OF-PERIOD FLOW
                          TIME    RAIN    EXCS     COMP Q
                             1    0.0     0.0         0.
                             2    0.0     0.0         0.
                             3    0.01    0.01        1.
                             4    0.05    0.05        8.
                             5    0.07    0.07       31.
                             6    0.11    0.11       82.
                             7    0.12    0.12      167.
                             8    0.15    0.15      285.
                             9    0.19    0.19      429.
```

Sheet 3 of 13

10	0.20	0.20	552.
11	0.21	0.21	767.
12	0.23	0.23	947.
13	0.23	0.23	1121.
14	0.25	0.25	1282.
15	0.25	0.25	1427.
16	0.32	0.32	1566.
17	0.33	0.33	1705.
18	0.34	0.34	1859.
19	0.34	0.34	2018.
20	0.35	0.35	2182.
21	0.35	0.35	2331.
22	0.35	0.35	2459.
23	0.64	0.64	2590.
24	0.93	0.93	2799.
25	0.94	0.94	3178.
26	0.95	0.95	3762.
27	0.96	0.96	4483.
28	0.96	0.96	5215.
29	0.97	0.97	5861.
30	0.98	0.98	6382.
31	0.35	0.35	6728.
32	0.36	0.36	6830.
33	0.35	0.35	6619.
34	0.35	0.35	6129.
35	0.36	0.36	5507.
36	0.35	0.35	4909.
37	0.36	0.36	4415.
38	0.31	0.31	4044.
39	0.27	0.27	3751.
40	0.28	0.28	3498.
41	0.27	0.27	3266.
42	0.28	0.28	3055.
43	0.27	0.27	2876.
44	0.27	0.27	2731.
45	0.27	0.27	2616.
46	0.0	0.0	2503.
47	0.0	0.0	2340.
48	0.0	0.0	2086.
49	0.0	0.0	1747.
50	0.0	0.0	1383.
51	0.0	0.0	1054.
52	0.0	0.0	787.
53	0.0	0.0	587.
54	0.0	0.0	438.
55	0.0	0.0	324.
56	0.0	0.0	241.
57	0.0	0.0	179.
58	0.0	0.0	134.
59	0.0	0.0	100.
60	0.0	0.0	74.
61	0.0	0.0	54.
62	0.0	0.0	40.
63	0.0	0.0	29.
64	0.0	0.0	20.
65	0.0	0.0	14.
66	0.0	0.0	9.
67	0.0	0.0	6.
68	0.0	0.0	3.
69	0.0	0.0	1.
70	0.0	0.0	0.
71	0.0	0.0	0.
72	0.0	0.0	0.
73	0.0	0.0	0.
74	0.0	0.0	0.
75	0.0	0.0	0.

76	0.0	0.0	0.
77	0.C	0.0	0.
78	0.0	0.0	0.
79	0.C	0.0	0.
80	0.0	0.0	0.
81	0.C	0.0	0.
82	0.0	0.0	0.
83	0.C	0.0	0.
84	0.0	0.0	0.
85	C.C	0.0	0.
86	0.0	0.0	0.
87	0.C	0.0	0.
88	0.0	0.0	0.
89	0.C	0.0	0.
90	0.0	0.0	0.
91	0.C	0.0	0.
92	0.0	0.0	0.
93	0.C	0.0	0.
94	0.0	0.0	0.
95	0.C	0.0	0.
96	0.0	0.0	0.
97	0.C	0.0	0.
98	0.0	0.0	0.
99	0.C	0.0	0.
100	0.0	0.0	0.
SUM	16.48	16.48	136655.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	6830.	2987.	1367.	1367.	136651.
INCHES		16.35	16.62	16.62	16.62
AC-FT		1482.	1507.	1507.	1507.

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Sheet 5 of 13

HYDROGRAPH ROUTING

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME
1	1	0	0	2	1	0

ROUTING DATA

QLOSS	CLOSS	AVG	IRES	ISAME
0.0	0.0	0.0	1	0

NSTPS	NSTDL	LAG	AMSKK	X	TSK	STORA
1	0	0	0.0	0.0	0.0	0.

STORAGE=	0.	67.	205.	305.	330.	375.	520.	590.	660.	800.
OUTFLOW=	0.	20.	160.	345.	530.	600.	774.	1000.	1526.	2740.

TIME	EOP STOR	AVG IN	EOP OUT
1	0.	0.	0.
2	0.	0.	0.
3	0.	0.	0.
4	0.	4.	0.
5	0.	20.	0.
6	1.	56.	0.
7	2.	125.	1.
8	5.	226.	1.
9	9.	357.	3.
10	14.	511.	4.
11	22.	680.	6.
12	31.	857.	9.
13	42.	1034.	13.
14	55.	1201.	17.
15	70.	1356.	23.
16	86.	1498.	40.
17	104.	1636.	57.
18	123.	1780.	76.
19	143.	1937.	97.
20	165.	2100.	119.
21	188.	2256.	143.
22	213.	2395.	175.
23	239.	2524.	222.
24	266.	2694.	272.
25	295.	2988.	327.
26	329.	3470.	527.
27	368.	4122.	590.
28	415.	4849.	648.
29	468.	5538.	712.
30	527.	6122.	804.
31	589.	6555.	1063.
32	650.	6779.	1461.
33	706.	6724.	1921.
34	752.	6374.	2327.
35	789.	5818.	2645.
36	816.	5208.	2879.
37	835.	4662.	3041.
38	847.	4229.	3150.
39	855.	3897.	3218.
40	859.	3624.	3255.
41	861.	3382.	3267.
42	860.	3160.	3257.

PEAK EL. = 263.45±
261.0 TOD
2.45 OVERTOPPED

43	857.	2966.	3230.
44	852.	2803.	3191.
45	847.	2673.	3144.
46	840.	2560.	3091.
47	833.	2421.	3030.
48	825.	2213.	2955.
49	814.	1916.	2861.
50	800.	1565.	2742.
51	784.	1218.	2603.
52	767.	920.	2450.
53	748.	687.	2289.
54	725.	512.	2127.
55	711.	381.	1968.
56	693.	283.	1814.
57	676.	210.	1668.
58	660.	156.	1530.
59	645.	117.	1430.
60	631.	87.	1336.
61	618.	64.	1247.
62	605.	47.	1164.
63	593.	34.	1085.
64	581.	24.	1030.
65	571.	17.	985.
66	560.	12.	941.
67	550.	7.	899.
68	540.	4.	859.
69	531.	2.	820.
70	522.	1.	784.
71	514.	0.	767.
72	505.	0.	756.
73	497.	0.	747.
74	486.	0.	737.
75	481.	0.	727.
76	473.	0.	717.
77	465.	0.	708.
78	457.	0.	699.
79	450.	0.	690.
80	442.	0.	681.
81	435.	0.	672.
82	427.	0.	663.
83	420.	0.	654.
84	413.	0.	645.
85	406.	0.	637.
86	399.	0.	629.
87	392.	0.	620.
88	385.	0.	612.
89	378.	0.	604.
90	372.	0.	596.
91	365.	0.	586.
92	359.	0.	577.
93	353.	0.	568.
94	346.	0.	559.
95	340.	0.	551.
96	334.	0.	542.
97	328.	0.	524.
98	323.	0.	481.
99	318.	0.	443.
100	313.	0.	407.
SUM			108440.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	3267.	1968.	1084.	1084.	108440.
INCHES		10.77	13.19	13.19	13.19
AC-FT		976.	1196.	1196.	1196.

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RUNOFF SUMMARY, AVERAGE FLOW

		PEAK	6-HOUR	24-HOUR	72-HOUR	AREA
HYDROGRAPH AT	1	6830.	2987.	1367.	1367.	1.70
ROUTED TO	1	3267.	1968.	1084.	1084.	1.70

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½ PMF

49.5 % PMF

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HEC-1 VERSION DATED JAN 1973

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                         PINES LAKE
                         NEW JERSEY
                         DAM ANALYSIS    0.5 PMF

                         JOB SPECIFICATION
           NQ    NHR   NMIN   IDAY    IHR   IMIN METRC   IPLT   IPRT NSTAN
          100      0      8      0      0      0      0      0      0      0
                               JOPER     NWT
                                  3        0



     **********      **********      **********      **********      **********

                         SUB-AREA RUNOFF COMPUTATION
           ISTAQ    ICOMP    IECON    ITAPE     JPLT     JPRT    INAME
               1        0        0        0        2        1        0

                         HYDROGRAPH DATA
     IHYDG    IUHG   TAREA    SNAP   TRSDA   TRSPC   RATIO   ISNOW   ISAME   LOCAL
         0      -1    1.70     0.0     0.0     0.0     0.0       0       0       0

                         PRECIP DATA
                    NP    STORM      DAJ      DAK
                    45      0.0      0.0      0.0
                         PRECIP PATTERN
   0.0      0.0      0.0      0.0      0.0      0.02     0.03     0.04     0.06     0.07
   0.07     0.09     0.11     0.10     0.11     0.13     0.15     0.14     0.16     0.16
   0.16     0.17     0.30     0.45     0.47     0.45     0.47     0.49     0.50     0.50
   0.17     0.19     0.19     0.19     0.19     0.19     0.19     0.16     0.14     0.14
   0.14     0.14     0.14     0.14     0.15

                         LOSS DATA
     STRKR   DLTKR   RTIOL   ERAIN   STRKS   RTIOK   STRTL   CNSTL   ALSMX   RTIMP
       0.0     0.0    1.00     0.0     0.0    1.00     0.0     0.0     0.0     0.0

                         GIVEN UNIT GRAPH, NUHGQ=   25
    92.      344.     736.    1092.    1227.    1129.     920.     689.     515.     393.
   295.      221.     160.     121.      92.      69.      52.      40.      31.      23.
    17.       13.      10.       6.       5.
             UNIT GRAPH TOTALS    8292. CFS OR 1.01 INCHES OVER THE AREA

                         RECESSION DATA
           STRTQ=     0.0     QRCSN=     0.0     RTIOR= 1.00

                         END-OF-PERIOD FLOW
                    TIME   RAIN    EXCS     COMP Q
                       1   0.0     0.0          0.
                       2   0.0     0.0          0.
                       3   0.0     0.0          0.
                       4   0.0     0.0          0.
                       5   0.0     0.0          0.
                       6   0.02    0.02         2.
                       7   0.03    0.03        10.
                       8   0.04    0.04        29.
                       9   0.06    0.06        63.
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Sheet 7 of 13

10	0.07	0.07	114.
11	0.07	0.07	178.
12	0.09	0.09	251.
13	0.11	0.11	329.
14	0.10	0.10	411.
15	0.11	0.11	495.
16	0.13	0.13	577.
17	0.15	0.15	658.
18	0.14	0.14	739.
19	0.16	0.16	824.
20	0.16	0.16	908.
21	0.16	0.16	988.
22	0.17	0.17	1062.
23	0.30	0.30	1139.
24	0.45	0.45	1255.
25	0.47	0.47	1454.
26	0.45	0.45	1753.
27	0.47	0.47	2119.
28	0.49	0.49	2492.
29	0.50	0.50	2825.
30	0.50	0.50	3105.
31	0.17	0.17	3305.
32	0.19	0.19	3383.
33	0.19	0.19	3301.
34	0.19	0.19	3073.
35	0.19	0.19	2779.
36	0.19	0.19	2497.
37	0.19	0.19	2265.
38	0.16	0.16	2091.
39	0.14	0.14	1951.
40	0.14	0.14	1823.
41	0.14	0.14	1703.
42	0.14	0.14	1589.
43	0.14	0.14	1491.
44	0.14	0.14	1411.
45	0.15	0.15	1350.
46	0.0	0.0	1293.
47	0.0	0.0	1214.
48	0.0	0.0	1087.
49	0.0	0.0	914.
50	0.0	0.0	725.
51	0.0	0.0	554.
52	0.0	0.0	414.
53	0.0	0.0	308.
54	0.0	0.0	230.
55	0.0	0.0	171.
56	0.0	0.0	127.
57	0.0	0.0	94.
58	0.0	0.0	70.
59	0.0	0.0	52.
60	0.0	0.0	39.
61	0.0	0.0	28.
62	0.0	0.0	21.
63	0.0	0.0	15.
64	0.0	0.0	11.
65	0.0	0.0	7.
66	0.0	0.0	5.
67	0.0	0.0	3.
68	0.0	0.0	2.
69	0.0	0.0	1.
70	0.0	0.0	0.
71	0.0	0.0	0.
72	0.0	0.0	0.
73	0.0	0.0	0.
74	0.0	0.0	0.
75	0.0	0.0	0.

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76	0.0	0.0	0.
77	0.0	0.0	0.
78	0.0	0.0	0.
79	0.0	0.0	0.
80	0.0	0.0	0.
81	0.0	0.0	0.
82	0.0	0.0	0.
83	0.0	0.0	0.
84	0.0	0.0	0.
85	0.0	0.0	0.
86	0.0	0.0	0.
87	0.0	0.0	0.
88	0.0	0.0	0.
89	0.0	0.0	0.
90	0.0	0.0	0.
91	0.0	0.0	0.
92	0.0	0.0	0.
93	0.0	0.0	0.
94	0.0	0.0	0.
95	0.0	0.0	0.
96	0.0	0.0	0.
97	0.0	0.0	0.
98	0.0	0.0	0.
99	0.0	0.0	0.
100	0.0	0.0	0.
SUM	7.86	7.86	65177.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	3383.	1433.	652.	652.	65175.
INCHES		7.84	7.93	7.93	7.93
AC-FT		711.	719.	719.	719.

OVN

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Sheet 11 of 13

HYDROGRAPH ROUTING

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME
1	1	0	0	2	1	0

ROUTING DATA

QLOSS	CLOSS	AVG	IRES	ISAME
0.0	0.0	0.0	1	0

NSTPS	NSTDL	LAG	AMSKK	X	TSK	STORA
1	0	0	0.0	0.0	0.0	0.

STORAGE=	0.	67.	205.	305.	330.	375.	520.	590.	660.	800.
OUTFLOW=	0.	20.	160.	345.	536.	600.	774.	1066.	1526.	2740.

TIME	EOP STOR	AVG IN	EOP OUT
1	0.	0.	0.
2	0.	0.	0.
3	0.	0.	0.
4	0.	0.	0.
5	0.	0.	0.
6	0.	1.	0.
7	0.	6.	0.
8	0.	19.	0.
9	1.	46.	0.
10	2.	89.	1.
11	3.	146.	1.
12	6.	214.	2.
13	9.	290.	3.
14	13.	370.	4.
15	18.	453.	5.
16	24.	536.	7.
17	30.	617.	9.
18	38.	699.	11.
19	46.	782.	14.
20	56.	866.	17.
21	66.	948.	20.
22	77.	1025.	30.
23	89.	1100.	42.
24	101.	1197.	55.
25	116.	1354.	69.
26	133.	1603.	86.
27	153.	1936.	107.
28	177.	2305.	131.
29	205.	2658.	160.
30	235.	2965.	216.
31	268.	3205.	276.
32	301.	3344.	338.
33	333.	3342.	541.
34	362.	3187.	582.
35	388.	2926.	615.
36	410.	2638.	642.
37	429.	2331.	665.
38	446.	2178.	685.
39	460.	2021.	702.
40	473.	1887.	718.
41	485.	1763.	732.
42	495.	1648.	744.

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43	503.	1540.	754.	
44	511.	1451.	763.	
45	518.	1381.	771.	
46	524.	1322.	789.	
47	525.	1253.	810.	
48	532.	1150.	826.	
49	534.	1000.	833.	← Peak Elev. 261.1±
50	534.	820.	833.	
51	532.	639.	824.	261.0 TOD
52	528.	484.	809.	
53	524.	361.	789.	
54	518.	269.	771.	0.1 overtopped
55	512.	200.	764.	
56	505.	149.	756.	
57	498.	110.	747.	49.5% PMF
58	491.	82.	739.	
59	483.	61.	730.	
60	476.	46.	721.	
61	468.	34.	712.	
62	461.	25.	703.	
63	453.	18.	694.	
64	446.	13.	685.	
65	438.	9.	676.	
66	431.	6.	667.	
67	424.	4.	658.	
68	416.	2.	650.	
69	409.	1.	641.	
70	402.	0.	633.	
71	395.	0.	625.	
72	389.	0.	616.	
73	382.	0.	608.	
74	375.	0.	600.	
75	369.	0.	591.	
76	362.	0.	582.	
77	356.	0.	573.	
78	350.	0.	564.	
79	343.	0.	555.	
80	337.	0.	546.	
81	331.	0.	538.	
82	326.	0.	502.	
83	320.	0.	462.	
84	315.	0.	425.	
85	311.	0.	390.	
86	307.	0.	359.	
87	303.	0.	341.	
88	299.	0.	334.	
89	296.	0.	328.	
90	292.	0.	321.	
91	288.	0.	314.	
92	285.	0.	308.	
93	282.	0.	302.	
94	278.	0.	296.	
95	275.	0.	290.	
96	272.	0.	284.	
97	269.	0.	278.	
98	266.	0.	273.	
99	263.	0.	267.	
100	260.	0.	262.	
SUM			41710.	

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	833.	702.	417.	417.	41710.
INCHES		3.84	5.07	5.07	5.07
AC-FT		348.	460.	460.	460.

sheet 13 of

RUNOFF SUMMARY, AVERAGE FLOW

		PEAK	6-HOUR	24-HOUR	72-HOUR	AREA
HYDROGRAPH AT	1	3383.	1433.	652.	652.	1.70
ROUTED TO	1	833.	702.	417.	417.	1.70